PRESTRESSED CONCRETE INSTITUTE

Vol. 1 Dec. 1956 No. 3

JOURNAL

OF PRESTRESSED CONCRETE INSTITUTE

IN THIS ISSUE

Technical Papers — 2nd Annual Convention

May 16-17-18, 1956

What Is PCI? ..... J. Rex Farrior, Jr.

The Characteristics of Prestressed Concrete Under Repetitive Loading ..... C. E. Ekberg, Jr.

Structural and Economic Advantages of Cast-in-Place Prestressed Concrete for Large Span Bridges ..... Curzon Dobell

The Physical Properties and Methods of Testing Prestressed Concrete Wire and Strand ..... H. J. Godfrey

Economics of Prestressed Concrete Bridges ..... T. Y. Lin

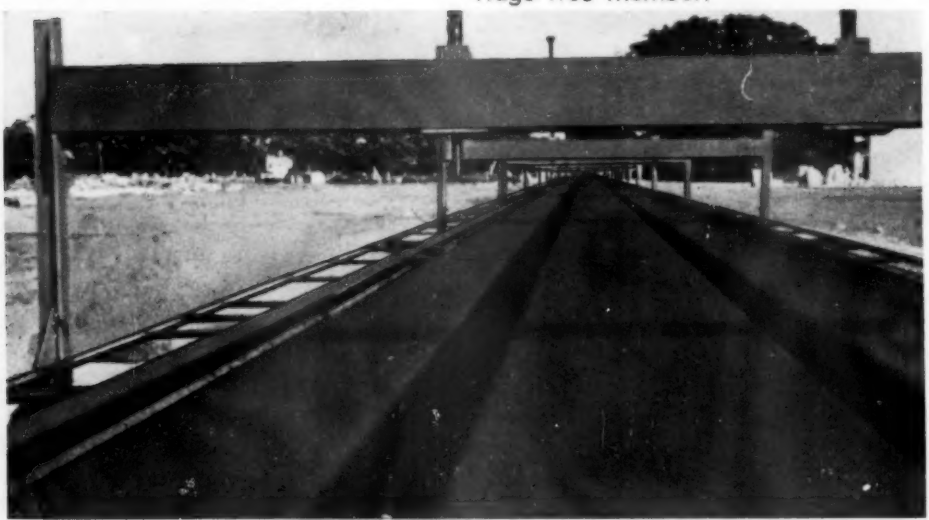
A Look at Prestressed Concrete Thus Far in California ..... Arthur L. Eillott

ONE WATCO FORM

LEADS TO ANOTHER

WATCO forms have proved themselves through dependable, profitable performance. That's why they are first choice with producers when increased production facilities are planned. WATCO forms are fabricated of first-quality steel in a variety of gauges. They are available in permanent, portable and interchangeable styles. For mass production of prestressed concrete structural units of all standard designs, install WATCO forms.

WATCO Double Tee forms feature a built-in extruded aluminum chamfer strip to provide sharper edges and eliminate chamfer strip installation for each pour. Two-piece construction with one weld on underside of flange means a more attractive, ridge-free member.



WRITE FOR DATA ON FORMS FOR

- Piling
- Double Tee Roof and Floor Slabs
- Beams
- Joists
- Girders

steel watco forms

Southern Prestressed Concrete Co., Pensacola, Florida, is now casting Double Tee roof and floor slabs in this new, 420'-0" long WATCO steel form.

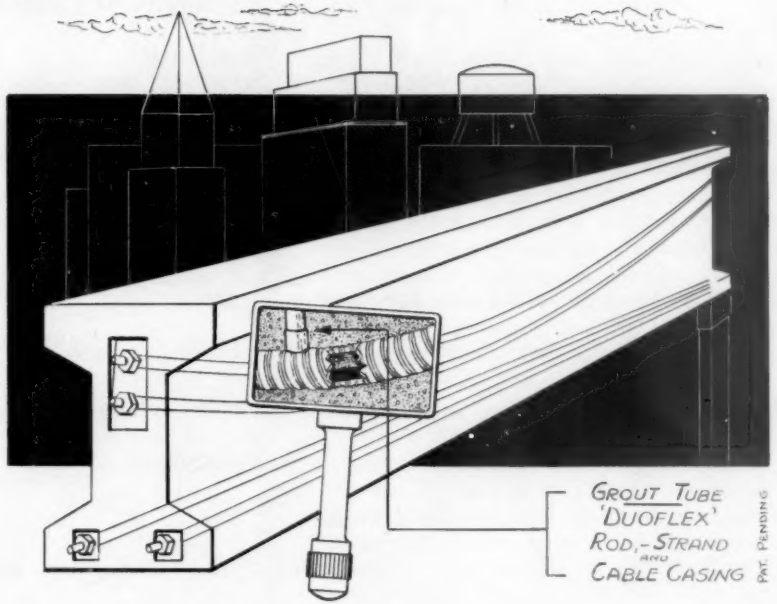
PLANT CITY

Welding & Tank Company

P. O. BOX 1308 PLANT CITY FLORIDA

BEST FOR POST-TENSIONING . . .

DUOFLEX



There is only one DUOFLEX casing specially designed for post-tensioning prestressed concrete used over bar, strand, or cable

DUOFLEX is designed for the greatest possible economy to keep your bids low—providing the best possible price for casing delivered to the job-site.

DUOFLEX casing has no peer. It is designed right and made right.
Specify DUOFLEX casing and Grout Connections and be right on all counts.

FLEXICO PRODUCTS, INC.

Flexible Metal Hose and Tubing for Industry

Metuchen, New Jersey

PRESTRESSED CONCRETE INSTITUTE

WORLD

Progress in prestressed concrete
will be the theme of a WORLD CONFERENCE
to be presented by the University of California
in cooperation with the Prestressed Concrete Institute
and other technical societies.
The 3rd Annual meeting of the PCI
will be held simultaneously with the Conference.

The 5-day Conference will bring together scientists,
engineers and manufacturers from the United States,
Canada and foreign countries in order that their knowledge and
experience may be pooled for the advancement of the
science and industry of prestressed concrete.

The dates of the Conference are:
July 29 through August 2, 1957.

The place: Fairmont Hotel, San Francisco, California.

Registration fee: $25.00 (U. S. Currency).

For reservation forms write to:
Department of Conferences & Special Activities,
University Extension, University of California,
Berkeley 4, California, U.S.A.

Make your reservation now to attend

this **CONFERENCE**

JOURNAL

Vol. 1 December 1956 No. 3

OFFICERS & DIRECTORS

1956-1957

J. Ashton Gray
President

Ben C. Gerwick, Jr.
Vice President

Douglas Cone
Secretary-Treasurer

DIRECTORS

Peter Verna, Jr.

Harold A. Price

Werner Rosch

Carroll Strohm, Jr.

Arthur C. Bruce

Dr. T. Y. Lin

Charles C. Zollman

George W. Ford

EXECUTIVE SECRETARY

Martin P. Korn

From the editor . . .

The primary functions of the PCI JOURNAL are: the exchange of ideas among the members of the Prestressed Concrete Institute; to further the knowledge, application, production and use of prestressed concrete. To achieve this goal, members from every phase of the industry should make frequent contributions, technical and non-technical, to the JOURNAL.

Items that would be of interest to our many readers include new and unusual applications, results of theoretical and experimental research, production problems, and transportation and construction problems involving extraordinary features. Photographs or drawings of structures having unusual features would also be welcome contributions.

The success of this publication depends on the interest and active participation of the members of the Institute and our readers. Let us hear from you often; we welcome any news of the prestressed concrete industry and your organizations.

The **JOURNAL** is published quarterly by the **PRESTRESSED CONCRETE INSTITUTE, INC.**, 425 N. E. 5th Street, Boca Raton, Florida. **TECHNICAL EDITOR:** Dr. A. M. Ozell, University of Florida, Gainesville, Florida. **ADVERTISING, SUBSCRIPTION** and **PUBLICATION OFFICE:** 3132 N. E. 9th Street, Fort Lauderdale, Florida. **DISTRIBUTION:** Available without cost to all members of the Prestressed Concrete Institute. Additional copies offered to members at $1.00 per copy. **SUBSCRIPTION PRICES:** Single copies: $2.00. Subscription rate: $6.00 per year for four issues. For subscriptions outside the continental limits of the United States, add 20% to cover handling and mailing.

Editorial contributions to the **JOURNAL** are welcomed, but publication cannot be guaranteed. All manuscripts should be submitted in duplicate and will be reviewed and approved by the Technical Editor prior to publication. Advertisements of products, services and materials allied to the prestressed concrete industry are welcomed in the **JOURNAL**. Address all inquiries concerning advertising rates and mechanical specifications to the Publication Office.

©

425 N. E. FIFTH STREET
BOCA RATON, FLORIDA

MEMBERSHIP QUALIFICATIONS

SCHEDULE OF DUES

Anyone engaged in the production of prestressed concrete, or in the production of materials or equipment allied to the prestressing industry; licensed architects and engineers; engineers and architects in training, and students enrolled in accredited schools of architecture and engineering, are invited to apply for membership in the Prestressed Concrete Institute. Membership classifications have been established as follows:

ACTIVE

Organizations and individuals actively engaged in the production of prestressed concrete products.
Annual Membership Dues. **$250.00**

ASSOCIATE

Organizations and individuals engaged in the production of materials and equipment allied to the prestressed concrete industry.
Annual Membership Dues. **$250.00**

PROFESSIONAL

Limited to registered architects and engineers.
Annual Membership Dues. **$ 25.00**

AFFILIATE

Limited to supervisory and technical employees, non-graduate.
Annual Membership Dues. **$ 15.00**

JUNIOR

Limited to architects and engineers in training.
Annual Membership Dues. **$ 15.00**

STUDENT

Limited to students of accredited architectural and engineering schools.
Annual membership dues, effective January 1, 1957 **$ 10.00**
(For student members outside continental U. S., Canada included) **$ 12.00**

Members receive the PCI JOURNAL, a quarterly technical publication, and PCItems, a monthly news bulletin, without additional cost, plus research bulletins and other information on prestressing as it becomes available.

PRESTRESSED CONCRETE INSTITUTE

425 N. E. FIFTH STREET
BOCA RATON, FLORIDA

APPLICATION FOR MEMBERSHIP IN PRESTRESSED CONCRETE INSTITUTE
(A non-profit corporation)

1. Name and address of company applying for membership:

_____

_____

_____

2. Name of person to represent company in Institute affairs:

_____

3. Type of membership applied for:

☐ Active (manufacturer)

I have done the following types of prestressing:

- ☐ Post-tensioning
- ☐ Pre-tensioning
- ☐ Combination of both of above
- ☐ Precasting
- ☐ On-the-job post-tensioning
- ☐ On-the-job pre-tensioning
- ☐ Design

☐ Associate (related business)

I am in the business of_____

_____

_____

☐ Professional (registered architect or engineer)
I hold certificate No._____ of the State of_____.

☐ Affiliate (limited to supervisory and technical employees, non-graduate)

☐ Junior (limited to architects and engineers in training)

☐ Student (limited to students of accredited schools of architecture and engineering)
Name of School_____

I understand the financial arrangements and am not to make any payment until approval of this application for membership.

Signed:_____

(cut along this line)

What is P.C.I.?

By J. Rex Farrior, Jr.
Legal Counselor for P. C. I
Tampa, Florida

P.C.I. is legally known as Prestressed Concrete Institute, Inc., a corporation not for profit organized and existing under the statutes of Florida, Chapter 617. The Institute is legally recognized as a scientific, charitable and benevolent association.

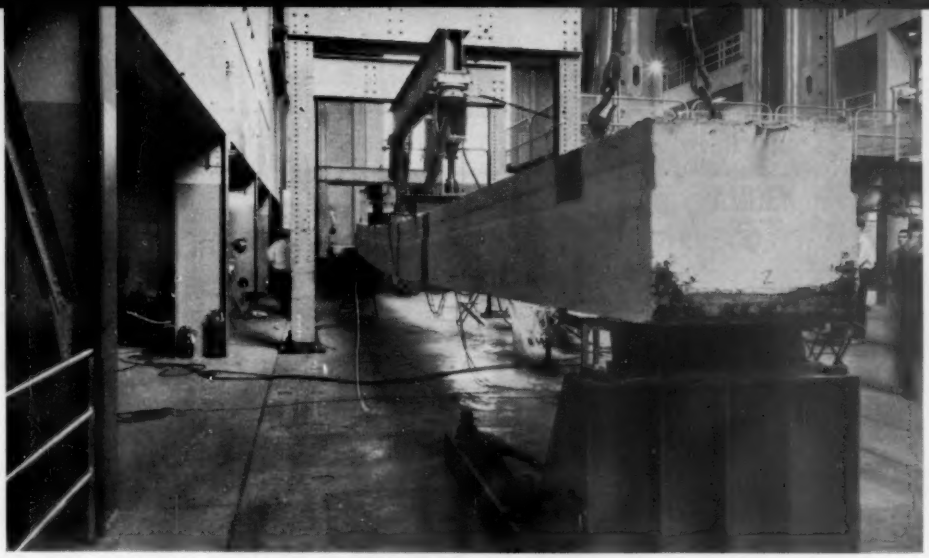
The original charter was approved by the Circuit Court at Tampa, Florida, on June 18, 1954. Officially, the object of the Institute is the formation of an association of manufacturers of prestressed concrete and of persons and companies in allied or related industries for the mutual exchange of information and ideas; and generally to do anything beneficial for its members and the prestressing business which does not violate the laws of Florida or the United States.

The charter provides for six classes of members: Active, for prestressed manufacturers; Associate, for related businesses; Professional, for architects and engineers; Junior, for architects and engineers in training; Student, for students enrolled in accredited architectural and engineering colleges, and Honorary, for such persons as the Board of Directors may wish to honor.

From a tax standpoint, the Institute enjoys an enviable position. Being a valid non-profit corporation, it is not required to pay any tax on its receipts, and therefore 100 per cent of its income can be devoted to research and other useful projects.

Fortunately, also, each member's dues and contributions are completely deductible by the member for income tax purposes.

Both tax factors therefore guarantee to the Institute's members more than full value for each dollar invested.



the characteristics of prestressed concrete under repetitive loading

C. E. Ekberg, Jr.
Associate Professor of Civil Engineering
Lehigh University

The main objective of this paper will be to show a possible approach to the problem of predicting the strength of prestressed concrete under repetitive loading. The problem is attacked by considering separately the characteristics of the steel and the concrete, utilizing certain test data which is available in the literature. Unfortunately, the test data which is referred to is not exactly suited to the type of prestressed concrete members which are now manufactured in the United States; however, we may still use it to show to some degree what may be expected of such members under repetitive loading.

Much work has been done with steel in connection with repetitive loading. All of us are familiar with the so-called S-N diagram for steel. Data is plotted from a series of experiments in which specimens are subjected to complete reversals of load until failure occurs. In Fig. 1 it is seen that as the applied completely reversed unit stress becomes smaller, the number of reversals N required to rupture the specimen becomes larger. Finally, we get to a point where the curve becomes approximately horizontal and the ordinate there is called the endurance limit of the material. In this case, it is seen that the endurance limit is approximately 19,000 psi. Accordingly, the material will rupture when subjected to several million cycles of completely reversed stress

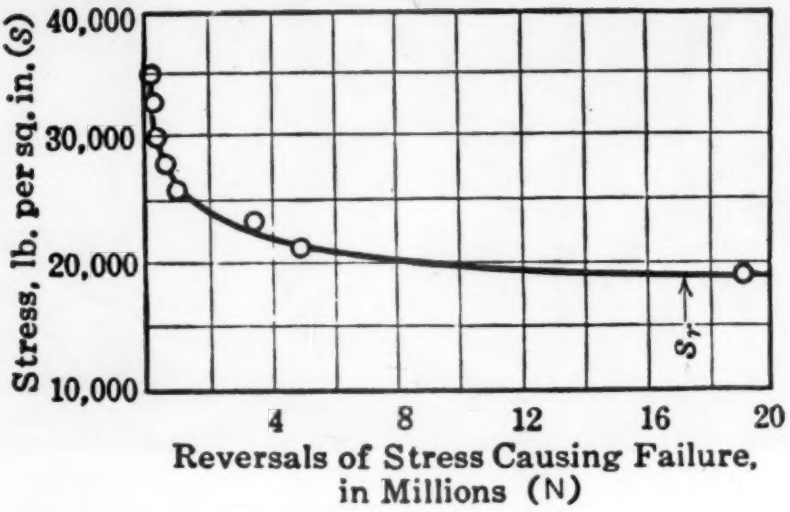


Fig. 1—An S-N Diagram for Steel.

at a level which is greater than 19,000 psi.

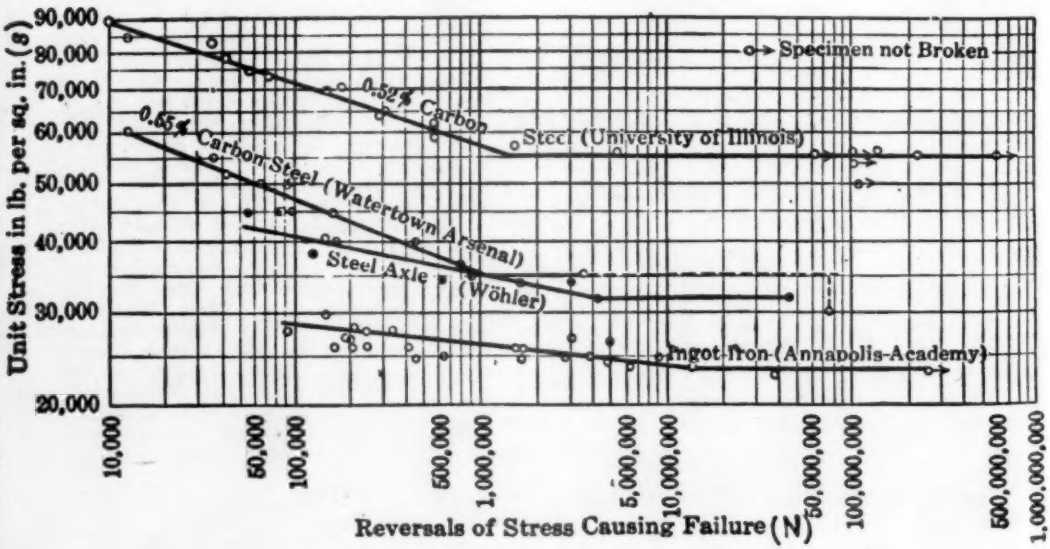
Values of the logarithms of S and N are plotted in Fig. 2 for various steels to show a large range of values in a rather compact fashion. It may be seen that for the higher carbon steels the curves level out at about 1 million cycles. The unit stress at these points of leveling out may be taken as the endurance limit.

The purpose of Fig. 3 is to show what is meant by laboratory-applied cyclic loading. The applied fiber stress on the specimen is plotted vertically and time is plotted horizontally. A tensile stress is considered in the positive sense upward and a compressive stress is negative downward. The first curve depicts a complete stress reversal of the type that we previously discussed. The second curve indicates stresses varying within a compressive range, and the third indicates stresses varying in the tensile range.

FAILURE ENVELOPE FOR STEEL WIRE

The heavy black curve in Fig. 4 represents a failure envelope for a 3mm. notched steel wire of a type used in Europe for prestressed concrete. This curve was obtained by Swiss investigators [1] who tested numerous specimens of the wire and determined various

Fig. 2—Logarithmic S-N Diagram for Steel.



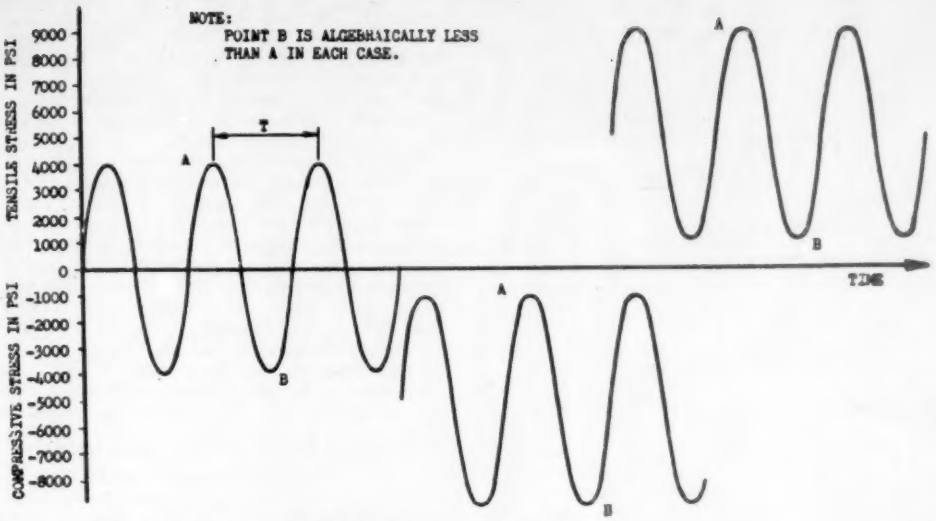


Fig. 3—Examples of Laboratory-Applied Repetitive Loading.

ranges of stress to cause failure at 1 million repetitions of load. One million repetitions is assumed to correspond approximately to the endurance limit. The procedure normally is to find the ultimate static strength of the wire first, and then to find by trial, different stress ranges to give failure at one million repetitions. With each critical stress range, the algebraically smaller stress is plotted horizontally and the larger vertically. For example, it is seen that failure of this wire would occur at one million cycles if the stress varied in tension from 246 ksi to 256 ksi, from 68 to 134 ksi, or from zero to 80 ksi. It seems logical that the smaller the range of stress, that is the smaller the difference between maximum and minimum, the greater is the stress intensity before failure

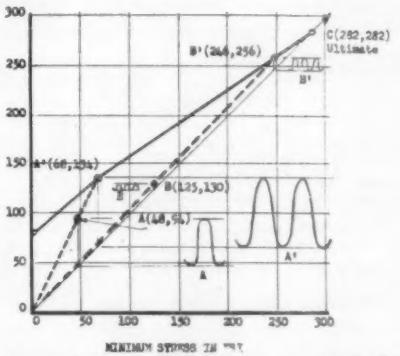


Fig. 4—Failure Envelope for 3mm. Notched Steel Wire.

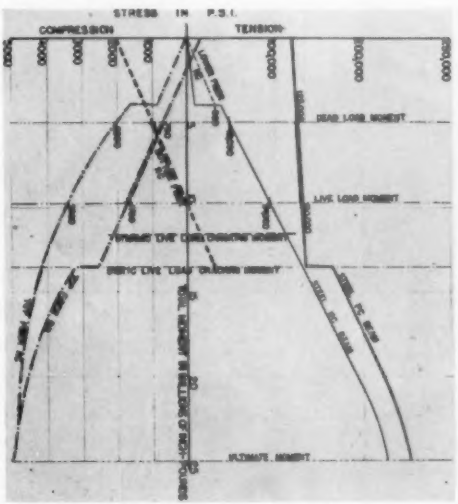


Fig. 5—Critical Stresses in Prestressed and Reinforced Concrete.

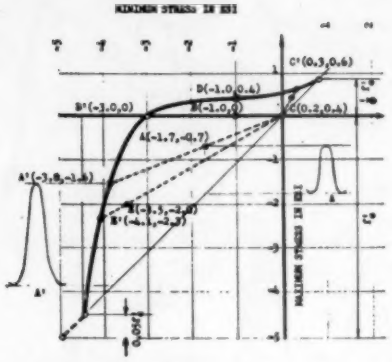


Fig. 6—Failure Envelope for Plain Concrete Prisms.

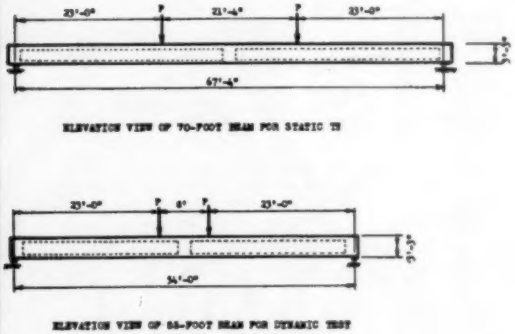


Fig. 7—Elevation Views of Test Beams.

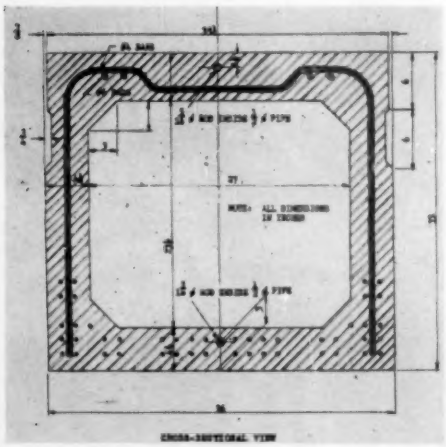


Fig. 8—Cross-Sectional View of Beams.

at one million cycles. To get a better physical picture of the meaning of the curve, we can draw a line from the origin at a 45 degree angle to a point of intersection at the extremity of the curve. Then it is possible to see directly what range of stress is involved as at point A′, where a horizontal line is drawn from A′ to determine the upper limit of stress and another horizontal line emanating from the point of intersection of a perpendicular dropped from A′ to the 45 degree line defines the lower stress limit. Similarly, it is seen that at B′ a much smaller stress range exists as defined by the two horizontal lines. This failure envelope is the result of original studies by Wohler and later investigations by Bauschinger.

Now let us use the failure envelope to determine a dynamic safety factor for the wire under working stress in a prestressed concrete beam. The next figure shows how the steel stress theoretically varied under working load in a 70-foot bridge beam recently tested at Lehigh.

ILLUSTRATION OF USE OF FAILURE ENVELOPE FOR WIRE

In Fig. 5, the maximum steel stresses are in tension and are plotted upward vs. the total moment in millions of inch pounds. The uppermost curve depicts the steel stresses in the 70 ft. pretensioned prestressed concrete bridge member, and it shows the stress before and after application of live load to be 125,000 psi and 130,000 psi respectively. At the static cracking moment the curve makes a jump and goes eventually to an ultimate stress of 250,000 psi. It is seen that the static safety factor is 250/130 = 1.92. Since the working range is between 125,000 and 130,000 psi, we will consider this as the critical range and returning to Fig. 4, we plot the point B (125,130) imagining, quite realistically, that there might be 1 million repetitions at this load. The point B is seen to fall below the failure envelope and hence will not cause failure. To obtain the dynamic safety factor at this ratio of minimum to maximum stress we need merely to project a line from the origin to point B until it strikes the failure envelope at B′. The coordinates of point B′ are seen to be 246 and 256 ksi for the abscissa and ordinate respectively. This means that at 1 million cycles of stress ranging between a maximum of 256 ksi and a minimum of 246 ksi for each cycle, we would get failure of our wire. The dynamic safety factor turns out to be 246/125 = 1.97 or 256/130 = 1.97 which is about the same as the static safety factor. Of course, we are ignoring the fact that in our actual beam we had twisted strand of ⅜ in. nominal diameter, and the failure envelope is representative only of 3 mm. notched steel wire. To get the exact picture, we would have to have the failure envelope for the ⅜ in. strand.

Now let us go back to Fig. 5. For the same high strength steel acting as conventional reinforcing in an identical 70 ft. beam, we find the stress varying under working

load from 48,000 psi to 94,000 psi. This example is very unrealistic but it will still be used for illustrative purposes. It is desired now to determine a dynamic safety factor against failure at 1 million cycles for this stress range. The static safety factor is 250/94 = 2.66. Returning to Fig. 4 we see the point A, representing the stress range for the conventional reinforcing, located well below the failure envelope. By projecting from the origin through A to A′ we find from the coordinates of A′ that failure would occur at 1 million cycles of stress oscillations between 68 ksi and 134 ksi. The dynamic safety factor here is 68/48 = 1.42 which is considerably less than the static safety factor. Thus, it appears that the pretensioned wire is better able to resist dynamic stresses than the same wire would be if used as conventional reinforcing.

FAILURE ENVELOPE FOR PLAIN CONCRETE

Fig. 6 shows failure envelope based on some extensive tests of plain concrete prisms[2]. The curve corresponds to the one previously shown for steel wire, except that here there is a passage through three different quadrants, thus making the situation somewhat more complicated. We will first examine the curve carefully to get the full significance from it. First we note that again the algebraically smaller stresses are plotted horizontally vs. the algebraically larger stresses vertically. If we look at the point C′ with coordinate 0.3 and 0.6 ksi, we visualize that failure for a 5,000 psi concrete would take place after 1 million cycles of tensile stress ranging between 300 and 600 psi. If we go into the second quadrant to the point D we see that the coordinates are (−1.0, 0.4) and we would expect failure to occur at 1 million cycles of stress ranging from 1000 psi in compression to 400 psi tension. At point B′, we see that failure would occur with 1 million stress oscillations between zero and 3000 psi compression. At point A′, we see that 1 million compressive stress oscillations between 3800 and 1600 psi would cause failure. The investigators found excessive creep to take place with loadings greater than 95 per cent of ultimate compressive strength, hence the dashed line at the lower left-hand corner.

A 45 degree line can be projected from the origin to aid in plotting actual stress ranges for the various loading cycles. For example, we can project out horizontally from A′ to define the upper limit of stress range, and from the point of intersection of a perpendicular dropped from A′ to the 45 degree line, draw another horizontal line to define the lower limit.

ILLUSTRATION OF USE OF FAILURE ENVELOPE FOR CONCRETE

Several examples will now be given to illustrate the use of the failure envelope. Let us go back to Fig. 5 and examine the stress relationship for the top and bottom fibers of the prestressed concrete beam. The top fiber stress is observed to vary from 700 psi to 1700 psi in compression under working load, and the bottom fiber stress ranges between 1000 psi and zero. The factor of safety based on static ultimate for the top fiber is 5000/1700 = 2.94. The margin of safety against cracking based on the rupture modulus is 800 psi. Now let us go back to Fig 6 and determine the corresponding dynamic safety factor and dynamic margin of safety for the top and bottom fibers respectively. First we plot point A based on the working stress range for the top fiber. Point A is plotted using the prescribed rule with the minimum stress of −1.7 ksi as abscissas and maximum stress of −0.7 ksi as an ordinate, and we find that it is well within the failure envelope. A line drawn from the origin is extended through A to the failure envelope at A′ and the coordinates of A′ are seen to be −3.8 ksi and −1.6 ksi. Thus we find a dynamic safety factor of 3.8/1.7 = 2.24 for the top fiber against the static safety factor of 2.94 previously calculated, and so the top fiber appears to be quite safe. For the lower fiber, we plot the point B with coordinates (−1.00,0) in the customary manner, and by projecting along the axis to B′ find a dynamic safety factor of 3.0. This dynamic safety factor of 3.0 for the lower fiber is a bit misleading because if any tension were introduced into the cycle the safety factor would be drastically reduced. For example, we can show that the safety factor would be cut to 2.0 if the stress range lies between 1000 psi compression and 150 psi tension. It is next desired to find the dynamic cracking moment as limited by the lower fiber. If it is assumed that the lower stress limit will remain at −1.00 ksi, the upper limit corresponding to the dynamic cracking stress will be found to be 0.4 ksi (see point D). Going back to Fig. 5 we can see that 400 psi tension on the bottom fiber corresponds to a moment of about 11.5 x 10^6 in. lb. The dynamic margin of safety against cracking for this increased moment is thus 400 psi or about half of the static margin of safety.

Returning to the conventionally reinforced beam, let us check the top fiber wherein the stress ranges between 2000 psi and 3500 psi. This corresponds to a static safety factor of 5.0/3.5 = 1.43. Looking at Fig. 6 we plot this stress range at point E (−3.5, −2.0). Using the customary procedure we get point E′ which yields a dynamic safety factor of 4.1/3.5 = 1.17 somewhat less than the static safety factor of 1.43, and considerably less than the dynamic factor of safety of 2.24 which was found for the top fiber of the prestressed beam.

One final example concerns diagonal tensile stresses. Let us suppose that in both beams the diagonal tension varies between 200 and 400 psi. This stress range is plotted at point C, and point C′ is found in the customary manner. Thus we arrive at a dynamic factor of safety against cracking after 1 million repetitions of 300/200 = 1.5. The static safety factor based on modulus of rupture is 800/400 = 2.00.

	Prestressed Beam	Conventionally Reinforced Beams
Working Range (psi)	125,000 to 130,000	48,000 to 94,000
Static S. F.	1.92	2.66
Dynamic S. F.	1.97	1.42
Static M. S. (psi)	120,000	156,000
Dynamic M. S. (psi)	45,000	26,000
Note: All dynamic failures would theoretically occur after 1 million repetitions		

Table 1 – Safety Factors and Margins of Safety for 3mm Steel Wire (Used for both Prestressed and Conventional Reinforced Concrete)

Table 1 and Table 2 summarize all of the results obtained from the failure envelopes. It should be noted that for computing the dynamic safety factors a common ratio of maximum to minimum stress was assumed for each case. For example, a ratio of 125:130 was assumed to occur at failure for the steel wire, and a ratio of 700:1700 was assumed to occur at failure for the top fiber of concrete.

FULL SCALE BEAM TESTS

INTRODUCTION

In the first part of this paper it was shown that there is a somewhat low dynamic margin of safety against cracking in the bottom fiber of a prestressed concrete beam as compared with the static margin of safety. According to Fig. 5 the static margin of safety is about twice as great as that of the same beam under repetitive loading. We will now discuss some interesting test results which tend to indicate that the smaller margin of safety is apparently of little consequence as far as the performance of the beam is concerned.

DESCRIPTION OF BEAMS

The two beams which will now be discussed were full-scale pretensioned concrete bridge members manufactured by the Concrete Products Company of America (a Division of American-Marietta of Pennsylvania) at their plant in Pottstown, Pennsylvania.* The beams were identical except for length – one beam being 70 ft. long and the other of 55 ft. length. The two beams are shown in elevation in Fig. 7 and the cross-section is depicted in Fig 8. Both beams were poured during the same operation and with the same mix.

Fig. 7 shows the elevation view of the two beams and the position of the test loads applied at the Fritz Engineering Laboratory. The dotted lines depict the extent of the rectangular hollows extending over each half of the members except for solid portions at end and midspan. It will be noted that the only difference in the two loading set-ups is that the shorter beam has a smaller maximum dead load moment (212 kip ft.) and there is only 8 ft. between load points. There is a maximum dead load moment of 328 kip ft. for the 70 ft. beam and a distance of 21 ft. 4 in. between load points.

	Prestressed Beam		Conventional	Diagonal Tension
	Top Fiber	Bottom Fiber	Top Fiber	
Working Range (psi)	700 to 1700	-1000 to 0	-2000 to 3500	200 to 400
Static S. F.	2.94		1.43	2.00
Dynamic S. F.	2.24	3.00*	1.17	1.5
Static M. S. (psi)	3300	800*	1500	400
Dynamic M. S. (psi)	1900	400*	500	180
*Safety factor or margin of safety against tensile cracking of concrete Note: All dynamic failures would theoretically occur after 1 million repetitions				

Table 2 – Safety Factors and Margins of Safety for Concrete

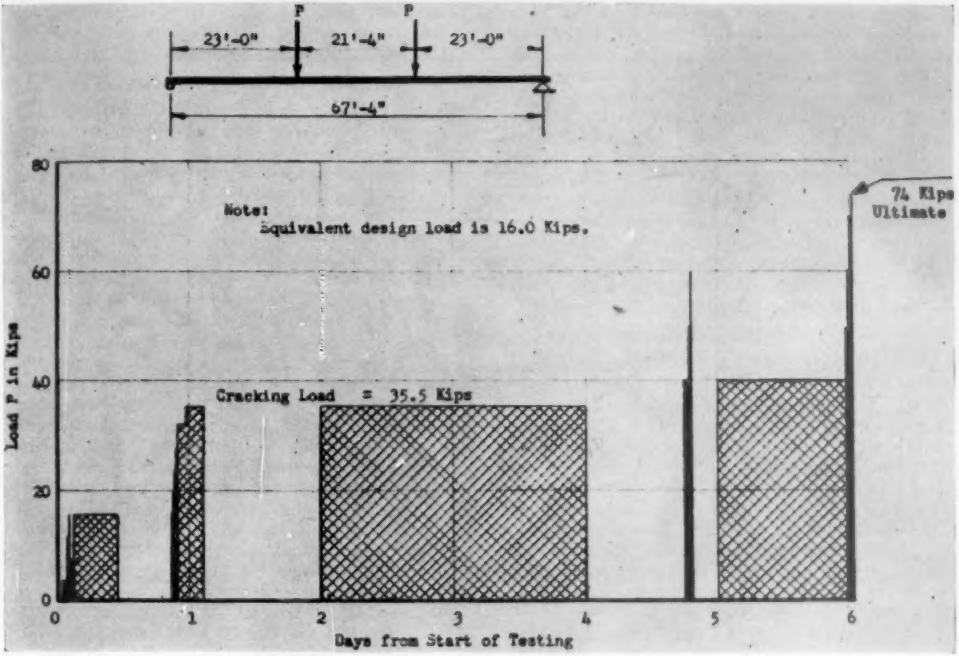


Fig. 9—Loading Sequence for Static Test.

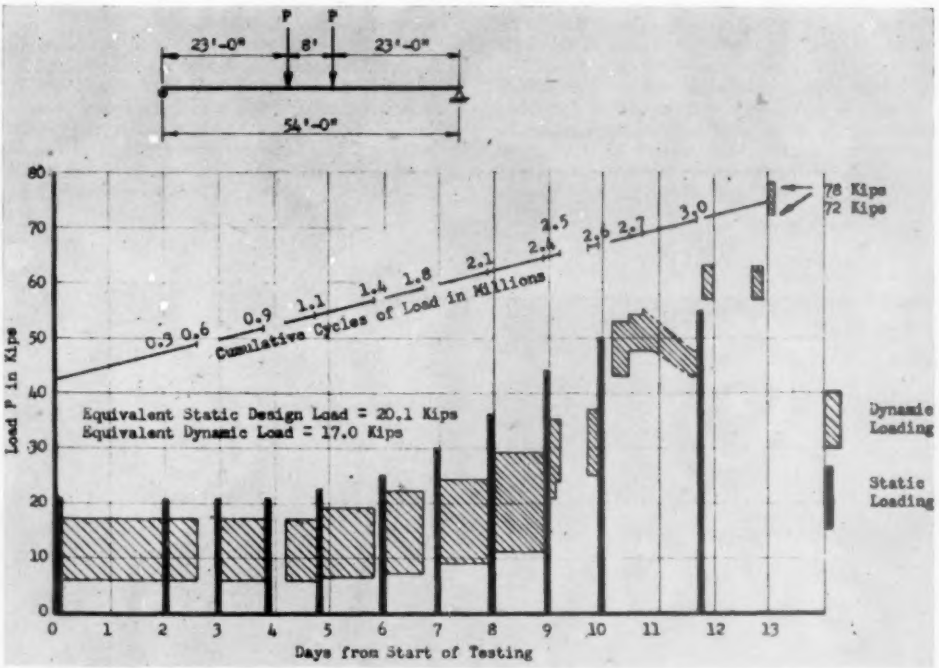


Fig. 10—Loading Sequence for Dynamic Test.

The beams were cast on the same prestressing bed so that the tendons, consisting of 46 stress relieved strands of ⅜ in. dia. (Fig. 8) could pass through the beams and give each the same prestressing force. Conventional reinforcing was used in the beams and consisted of inverted U-shaped stirrups at 8 in. centers, and 4 longitudinal bars were placed along the top fiber. Fig. 8 also shows two 5/16 in. dia. rods inside ½ in. dia. pipe near the top and bottom fibers for the special purpose of measuring the the total shortening and elongation of the member.

**These members are commercially available under the name "Amdek."*

DESCRIPTION OF LOADING

The loading diagram for the static test is shown in Fig. 9, and requires little explanation. The heavy black lines indicate the gradually applied loading and it is seen that there were four sustained loadings applied during the testing procedure as denoted by the cross-hatched areas. The equivalent design load was 16.0 kips per jack and the load at initial cracking was 35.5 kips per jack.

The loading diagram for the dynamic test is shown in Fig. 10. The heavy solid lines indicate the eleven static tests that were conducted in between periods of repetitive loading at 250 cycles per minute. The shaded portions denote the periods of repetitive loading and the cumulative cycles in millions are shown on the diagonal line across the top. The first repetitive loading was applied for 1.1 million cycles between a minimum loading of 5.5 kips per jack and a maximum of 17.0 kips per jack. The intensity of the load was then increased each day until at the eighth day it reached a range of from 11 to 29 kips per jack. For certain practical reasons, it was desirable to maintain the minimum jack load as given. The maximum dynamic load was calculated on the basis that the same maximum deflection would occur as for the corresponding static load. For instance, the 20.1 kip static load gives the same theoretical maximum deflection as the 17.0 to 5.5 kip dynamic load.

METHOD OF CRACK MEASUREMENT

All cracks were carefully marked and measured on one side of the beams only. This was done by means of a calibrated microscope which enabled the accurate measurement of crack widths to the nearest 0.001 in. as seen in Fig. 11. The readings on cracks were taken at a horizontal reference line located 2½ in. above the bottom fiber. In other words, the width of each crack at various static loads was recorded at the point where the crack crossed the horizontal reference line. In all cases, for both beams, the crack measurements were made during the static tests at the highest applied load of each run. For the 55 ft. beam, the first crack was not detected until the fourth static test after application of 900,000 load cycles. This crack measured approximately 0.001 in. in width and was located near mid-span. All cracks were nearly vertical in the region where the widths were measured and became less in width at points higher up the side of the beam. Fig. 12 shows the position and general appearance of the crack pattern of both the 55 ft. and 70 ft. beams.

RESULTS OF CRACK MEASUREMENTS

Figs. 13, 14 and 15 show the number of cracks, average crack width, and maximum crack width all as ordinates against total maximum applied moment as abscissa. The solid line represents the 70 ft. beam which was tested statically, and the broken line represents the 55 ft. beam.

Fig. 13 shows a trend which is typical of all three graphs in that the first crack appeared at rather low load for the 55 ft. beam, but there was some tendency toward convergence of the two curves at the higher loading. The total number of flexural cracks in the 70 ft. beam was found to be 83 at the highest moment of 1478 kips ft. at which

Fig. 11—Measurement of Crack Widths.

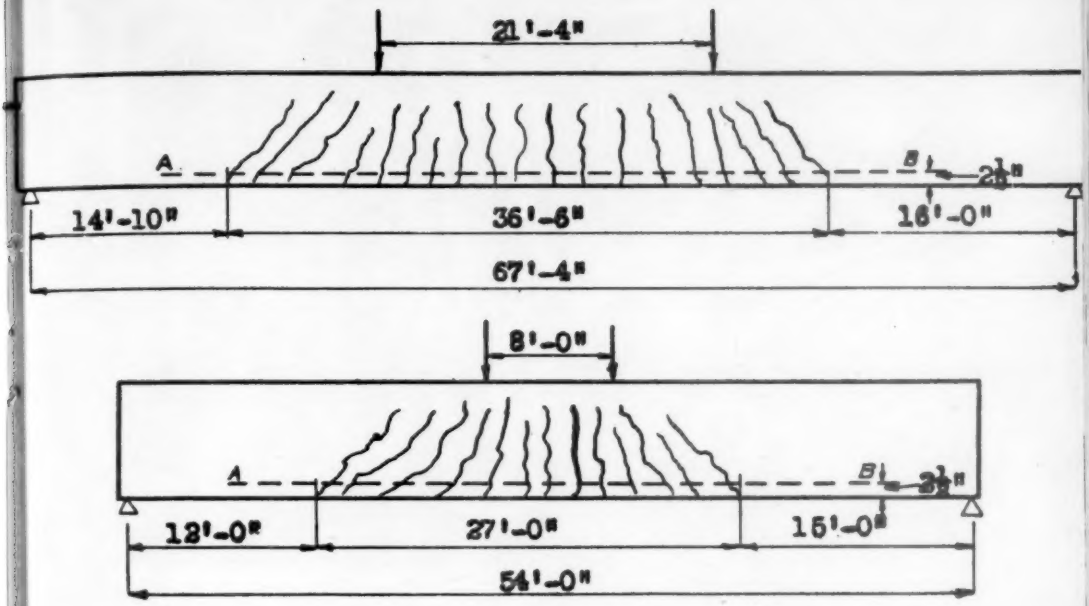


Fig. 12—Position and Appearance of Crack Pattern.

cracks were measured. The total design moment (dead load plus live load plus impact) is 752 kips ft. for the 70 ft. beam. For the case of the 55 ft. beam the number of cracks at 1478 kips ft. moment was found to be 52, however, it seems reasonable to believe that the number of cracks would have been about the same had the two beams been of the same length.

Fig. 14 shows the relationship between the average crack width and the total applied moment for both beams. It is seen that the two curves converge quite closely at the cracking moment of the 70 ft. beam, however, at the largest moment they have diverged somewhat to values of 0.007 in. and 0.005 in. for the 55 ft. beam and the 70 ft. beam respectively.

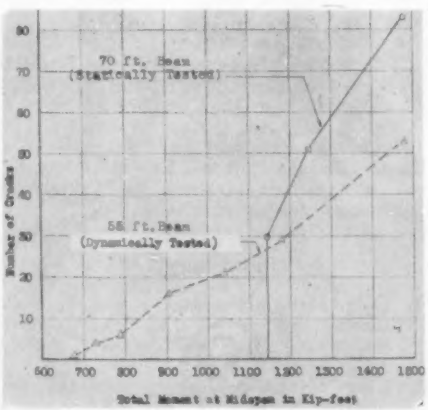


Fig. 13—Number of Cracks vs. Total Moment.

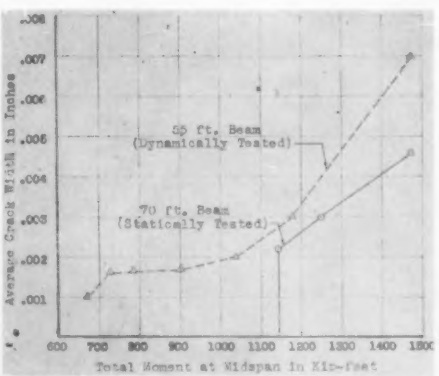


Fig. 14—Average Crack Width vs. Total Moment.

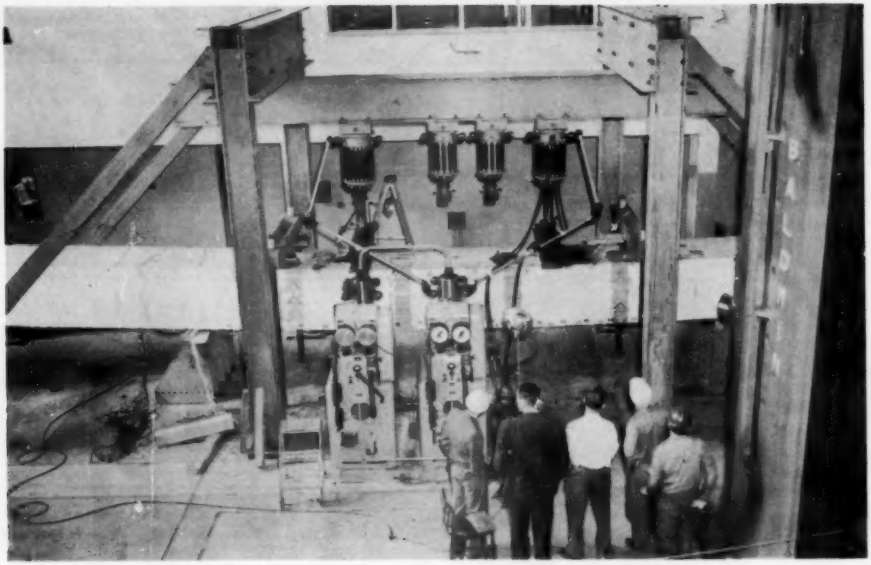


Fig. 16—View of Beam at High Load.

Fig 15 shows the relationship between the maximum crack width and the total moment. These curves reveal very strikingly the excellent behavior of the two beams, especially when one recalls that a crack is not considered to be detrimental as far as corrosion is concerned until it reaches a width in excess of 0.010 in. The critical width of 0.010 in. was reached at moments of 1280 and 1410 kips ft. for the 55 ft. beam and 70 ft. beam, respectively. Here again we note a close correlation between the behavior of the two beams under high moment.

As a final bit of evidence of the similarity of behavior of the two beams, it should be pointed out that the ultimate moments were practically the same. The total moments were 2030 kip-ft. and 2006 kip-ft. for the 70 ft. and 55 ft. beams, respectively. Fig. 16 shows the beams under test loading.

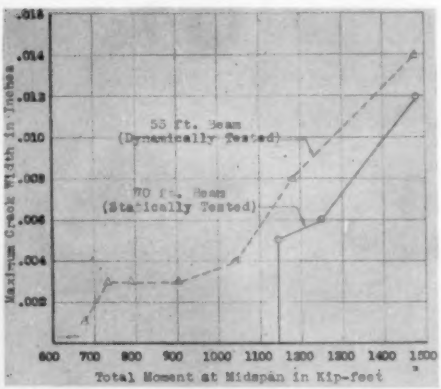


Fig. 15—Maximum Crack Width vs. Total Moment.

CONCLUSIONS AND SUMMARY

1. The essential purpose of this paper was to give a method for calculating the factor of safety of a beam which is subjected to repetitive loading, and to bring out the point that there is no single value to embrace all span lengths and conditions of prestressing.

2. It was shown that the dynamic ultimate strength and other characteristics of a properly designed prestressed concrete beam may not be greatly different from the static ultimate strength, even though the cracking loading is considerably lower for the dynamically applied load.

ACKNOWLEDGMENTS

The author wishes to thank the Concrete Products Company of America (a Division of American-Marietta Company of Pennsylvania) for allowing the use of data from the Lehigh tests on their bridge members. Thanks are also due Mr. Rene Walther, Research Associate, at the Fritz Engineering Laboratory, for his assistance in the preparation of this paper.

REFERENCES

1. Eidg. Materialprufungs — und Versuchsanstalt, Bericht Nr. 155, *Vorgespamiter Beton*, Zurich, March 1946, p. 79, Abb. 78.
2. Eidg. Materialprufungs — und Versuchsanstalt, Bericht Nr. 162, *Die Materialtechnischen und Probleme des Eisenbetons*, Zurich, 1950, p. 36, Abb. 55.

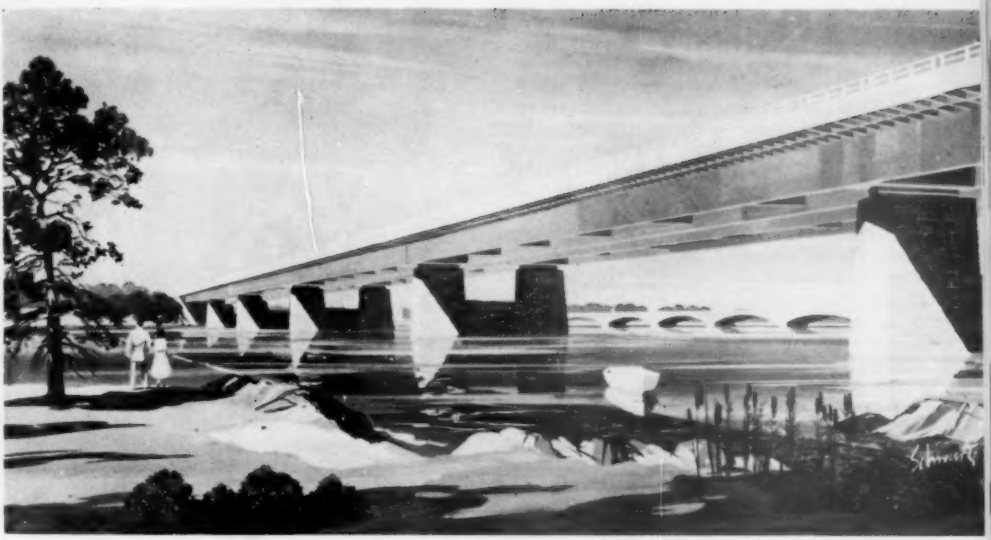


Fig. 22 — Perspective of imposing prestressed concrete bridge across the Potomac River in Washington, D. C.

STRUCTURAL AND ECONOMIC ADVANTAGES OF CAST-IN PLACE PRESTRESSED CONCRETE FOR LARGE SPAN BRIDGES . . .

By Curzon Dobell

M. A S C E, Consulting Engineer

It is most fitting to be holding this convention in the State of Florida which has done so much to advance the use of prestressed concrete for highway bridges under the able direction of the State Bridge Engineer, William E. Dean. Right after the war he foresaw the advantages of using prestressed concrete for many of the bridges required in meeting the backlog of highway construction. He recognized the need of standardizing design and construction procedures so that the theory of prestressing could be applied to assembly-line techniques to produce rapid and economical construction. The success of his efforts can be seen in many parts of the state where some 50 prestressed bridges have been built. They were not only cheaper than steel or reinforced concrete, with which they competed, but were also cheaper than most prestressed bridges built in other parts of the country.

In his paper to the Highway Research Board in January, Mr. Dean reported on the design, construction and cost of 15 of these bridges having a total length of 43,121 feet, of which 36,396 feet are built with post-

tensioned precast girders, 4,325 feet with pre-tensioned precast girders and 2,400 feet using combined pre-tensioning and post-tensioning. Span lengths varied from 25-72 ft. and superstructure costs from $2.95-$5.75 per square foot.

The large numbers of prestressed bridges constructed and planned in Florida are all based on the use of precast members, either pre-tensioned or post-tensioned. This preoccupation with precasting may be justified in Florida where the terrain and foundation conditions in most parts of the state lend themselves to short span trestle-type structures, but is not justified in other parts of the country where prestressing has yet to make a dent in the market for spans of 100-400 feet which are still monopolized by plate girder construction. Only cast-in-place prestressed construction can tap this lucrative market.

Transportation facilities, site conditions and lifting equipment, limits the length of precast members to about 80 feet, and at the upper limit it is often necessary, in order to minimize unit lifting weights, to use a number of closely-spaced members. This results in considerable waste of materials and excessive dead weight. Within the span lengths suitable for precasting prestressed members must compete with rolled structural steel shapes which are always erected at considerably lower cost per pound than longer span plate girders. We are promoting prestressed concrete in its least competitive area and neglecting longer span structures where the advantages of prestressing will produce much greater savings in capital cost and maintenance.

Just because precasting and prestressing were both born about twenty years ago it doesn't follow that they were born for each other or that they must forever be united in unholy deadlock until death do them part. Precasting and prestressing go well together for members up to 70-80 ft., but prestressing becomes increasingly important in long heavily-loaded members which can only be built with cast-in-place methods.

Much work in Europe, particularly in Germany, which has an economy similar to our own, shows us that the economic advantage of prestressed construction over the older methods of construction increases rapidly for longer cast-in-place spans up to the point where truss or suspension systems take over, and even for these prestressing is starting to be used with impressive savings in dead weight and cost.

During the past five years I have made annual trips to Europe where I have observed the advances being made with cast-in-place prestressed construction for long bridge and building spans. The few longer span cast-in-place prestressed structures which we have designed, or are designing, in this country show we can expect to develop, under our competitive bidding, much greater percentage savings in cost for such structures than can be obtained in the range of span suitable for precasting.

In 1949, when we designed the Walnut Lane Bridge in Philadelphia in collaboration with the late Professor Magnel, we were still laboring under the delusion that precasting and prestressing were complementary. We ended up using 13 girders, each weighing 150 tons, for the center span 160 feet long by 62.5 feet wide. Because we could not close Lincoln Drive under the bridge to traffic, we first considered casting the girders at one end of the bridge and moving them into position on a temporary cableway. This was abandoned because of the high cost of the cableway system. We finally resorted to building a narrow falsework, bridged over Lincoln Drive, on which the girders were cast one at a time and then jacked across the piers into position. As I will show later, it would have been much cheaper and quicker to have formed up the whole bridge and cast girders and slab monolithically in their final position.

Cast-in-place prestressed construction has many advantages over precasting for long spans, in particular:

a) Monolithic construction of webs, slab and diaphragms permits spacing girders to 10-30 ft. apart without consideration of their individual weight, and thus the most economical cross-section for concrete is obtained.
b) Transverse load distribution can be developed with mild steel instead of costly prestressing, and large concentrations of prestressing force in single casings can be used to greatly reduce the number of prestressing operations.
c) Skews, horizontal and vertical curves in the deck are readily accommodated.
d) Forming and grouting between precast members and separate casting of wearing surfaces are eliminated.
e) Large concrete sections may be used similar in dimension to what our general contractors are accustomed to forming and concreting for ordinary reinforced bridges of shorter spans.
f) Continuity over supports, when foundation conditions permit, increases economy of materials and cost.
g) Precasting plant, transport equipment and lifting equipment, needed for precast members, are not required.

What then are the arguments against this type of prestressed construction? They usually follow the same pattern we used to hear about any kind of prestressing. They include:

a) Difficulty of design, scarcity of trained engineers, and greater number of engineering man-hours required.
b) Problems of design and construction of falsework.
c) Greater difficulty in concrete quality-control.
d) Fear of restricted competitive bidding by general contractors.
e) Fear of possible patent infringements.
f) In general, fear of sticking the neck out

so far it becomes necessary to use the head instead of the handbook.

Let's consider some of these arguments –

It is true that with longer span cast-in-place structures, particularly when continuity is used, design calculations are somewhat more difficult and much more lengthy. Without considerable experience a good deal of trial and error is needed to find the most economical cross-section and balance of forces. It is always necessary to calculate stresses for both elastic theory and ultimate load. As no standards are available, it is necessary to make many more detail drawings and calculations. However, we are no longer pioneering in this field. Considerable research data is available and more than 100 cast-in-place multi-span prestressed bridges have been built. Dr. Fritz Leonhardt's new German book "SPANNBETON Fur Die Praxis," published in 1955, gives a good account of these works. This kind of design requires considerable study before it can be practiced with confidence or efficiency, but, there are now at least two consulting firms in this country which have experienced staffs for this work. Modesty precludes me from naming them.

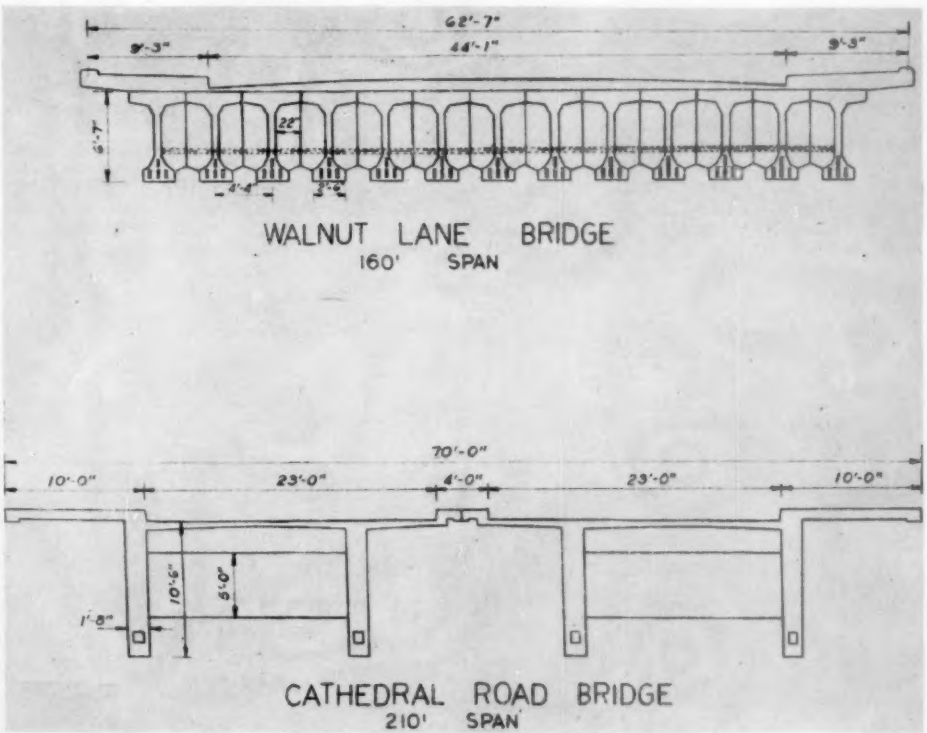
Because of the greater number of engineering man-hours required for such designs, owners must be prepared to pay consulting fees at least equal to A S C E Scale A, but, as will be shown, they will be rewarded by 20-40% savings in cost compared with structural steel, virtual elimination of maintenance cost and quicker construction time.

Just because the average highway department or general consultant may not have the experienced personnel to design such structures economically should be no reason for shunning its use in the face of such large savings in capital cost and maintenance. Owners do not hesitate to hire engineering specialists to design large suspension bridges and tunnels knowing that their experience will produce savings in construction cost many times greater than their fees. It is no different with the design of large prestressed structures which also requires specialization of design.

Now about falsework and forming, which is usually a source of argument. To permit uninterrupted concrete placing it is usually necessary to form up 3-4 spans, or series of spans for continuity. However, this is not as formidable as it may appear. When media strips are required the bridges can be designed symmetrical on each side of a longitudinal joint so that only half the deck needs to be formed at a time. Forms are progressively moved on a 2- to 3-week cycle to match up with concreting and prestressing cycles.

Three basic types of falsework can be

Fig. 1



used, the choice of which depends on many location factors:

1. Built up falsework in timber or steel. Where spans are below 120 ft. this type of falsework can be framed out from the piers without requiring intermediate supports. For longer spans intermediate temporary bents are required at about 60 ft. spacing which can be carried with 36 inch wide flange girders.
2. Self-centering falsework. This type of falsework is built around steel trusses of full span length. It is floated into position between piers. Then hoisted and secured in place with pipe pins through the piers and sand jacks.
3. Suspended falsework. Temporary towers are erected on piers to carry suspension cables from which hangs the falsework. It is necessary to provide adequate adjustment in the suspenders to maintain the elevation of the deck during concreting.

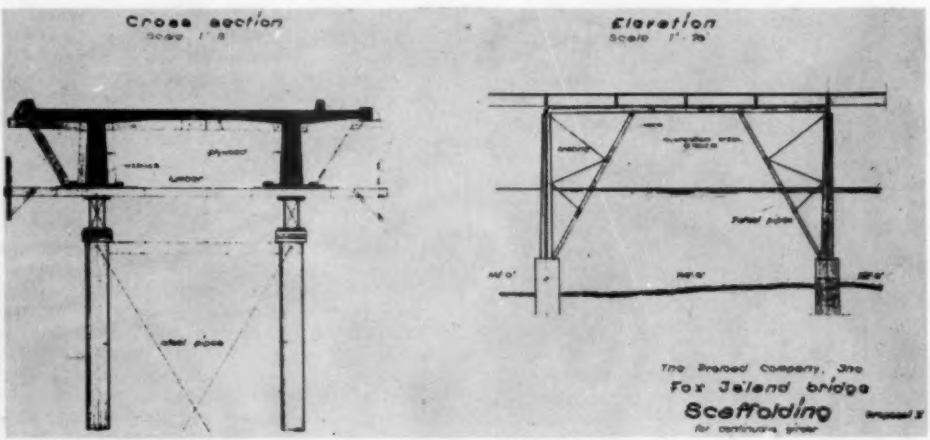
The fabrication cost of falsework by any one of these three methods will run from $2.50-$7.00 per sq. ft. which, of course, is divided by the number of re-uses. Cost of lowering, moving and resetting the falsework runs $0.50-$1.50 per square foot. The average cost for four or more re-uses, without regard to salvage, runs from $1.50-$3.00 per square foot. This is usually more than offset by the cost of precasting plant, transportation and erection of precast members, to say nothing of the other advantages enumerated earlier.

Another argument against cast-in-place is the difficulty in concrete control. This has to be viewed in two parts, (a) concrete manufacture and (b) concrete placing. The control of raw materials, batching and mixing of concrete is now a routine and standardized procedure whether the manufacture is done at a central mix plant or at the production yard. However, the control of placing and vibrating concrete is more difficult for cast-in-place than for precasting at a central plant. On the other hand, it is not so critical. Because concrete dimensions are usually more substantial than the minimum necessary for concrete placing, high concrete strength and stresses are not so important. For most cast-in-place construction a 28 day strength of 3500-4500 psi is usually the most economical. This is in the range of what is usual for large reinforced concrete spans and arches, and can be consistently maintained by experienced general contractors without resorting to unfamiliar techniques.

Now about competitive bidding by general contractors. Right here in Florida you have seen what contractors will do if given a chance. In the bidding to the Florida Roads Department on November 22, 1955, for the Manatee and Palma Sola bridges, eight bids were received for the prestressed concrete designs (which were low for both bridges), only two bids for the structural steel design and not a single bid on the reinforced concrete design. The same thing will be true with cast-in-place construction with which general contractors have wider experience than with precasting.

There is also the question of patents. There are now about 300 U. S. patents on prestressing but most of them are gadgets or relate to some minor construction procedure which can easily be avoided. However, as with all new engineering methods, it is important for the designer to keep abreast of patents so that he will not inadvertently stumble into infringement. In most cases, where a patented feature has any advantage, the designer can make an agreement with the patent owner for its use on a particular job at a modest price which can be stipulated in the specifications so that all contractors bid on an equal basis. If the patent owner is stubborn it is only necessary to make the primary design without the patented feature, details of which can be shown as an alternate with notice to contractors. The patent situation is not nearly

Fig. 2



as complicated as it was in the early days of flat slab design on which there were more than 1000 patents issued, but this did not prevent it becoming a standard procedure.

Now, to provide more graphic support for my theme,

ILLUSTRATIONS

Fig. 1 shows a comparison of cross-sections for precast and cast-in-place construction.

The main span of Walnut Lane is 160 ft. long by 62.5 ft. wide using 13 T-head girders each weighing 150 tons. In contrast is the cross-section of a study for the Cathedral Road bridge designed just three years later. It has three 210 ft. continuous spans 70 ft. wide. Because it is based on cast-in-place construction it is possible to carry the span with only 4 girders cast monolithically with the deck and diaphragms. The contrast in quantities per sq. ft. of deck is significant.

Walnut Lane —

Concrete .0915 c. y. Steel 16.5 lbs.

Cathedral Road —

Concrete .064 c. y. Steel 13.8 lbs.

The steel quantities include prestressing steel and anchors at 11.5 lbs. per sq. ft. for Walnut Lane and only 6.3 lbs. for Cathedral Road—a saving of nearly 50% in prestressing steel despite the larger span. The actual cost

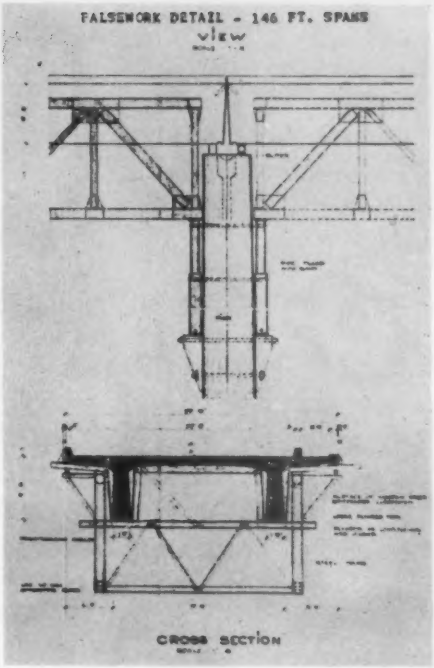
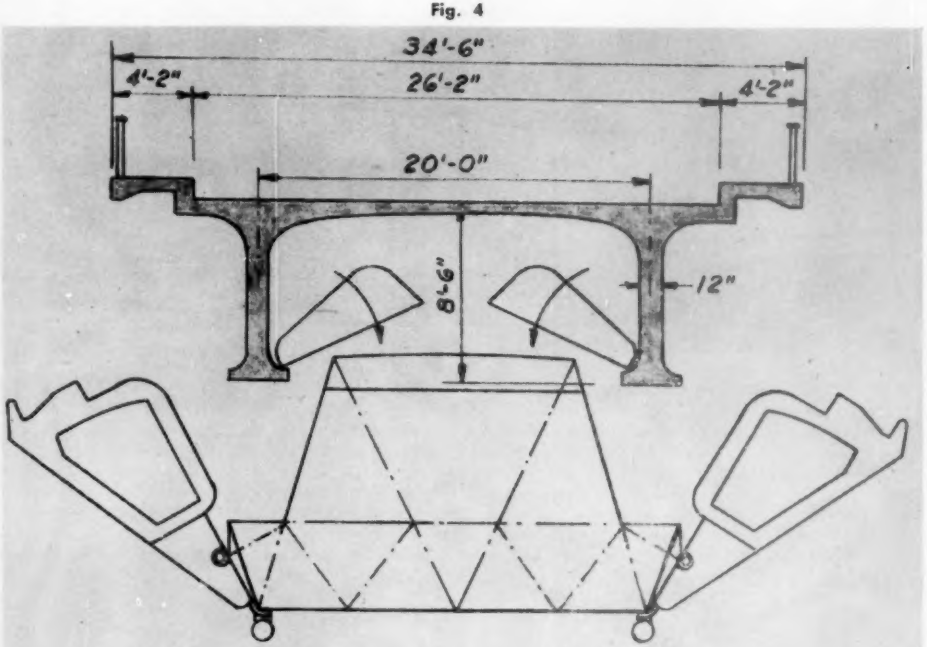


Fig. 3

Fig. 4



DETAILS OF SELF-CENTERING FALSEWORK
USED FOR 130 FT. SPANS
OF PORTO ALEGRE BRIDGE IN BRAZIL

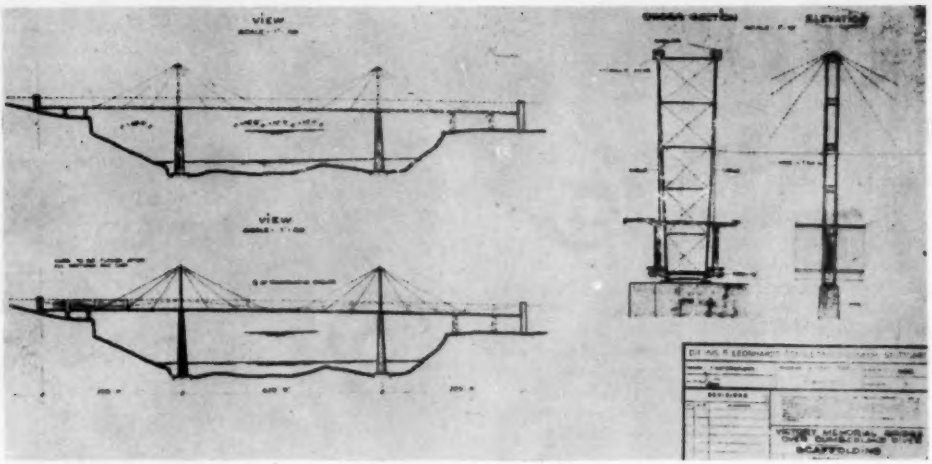


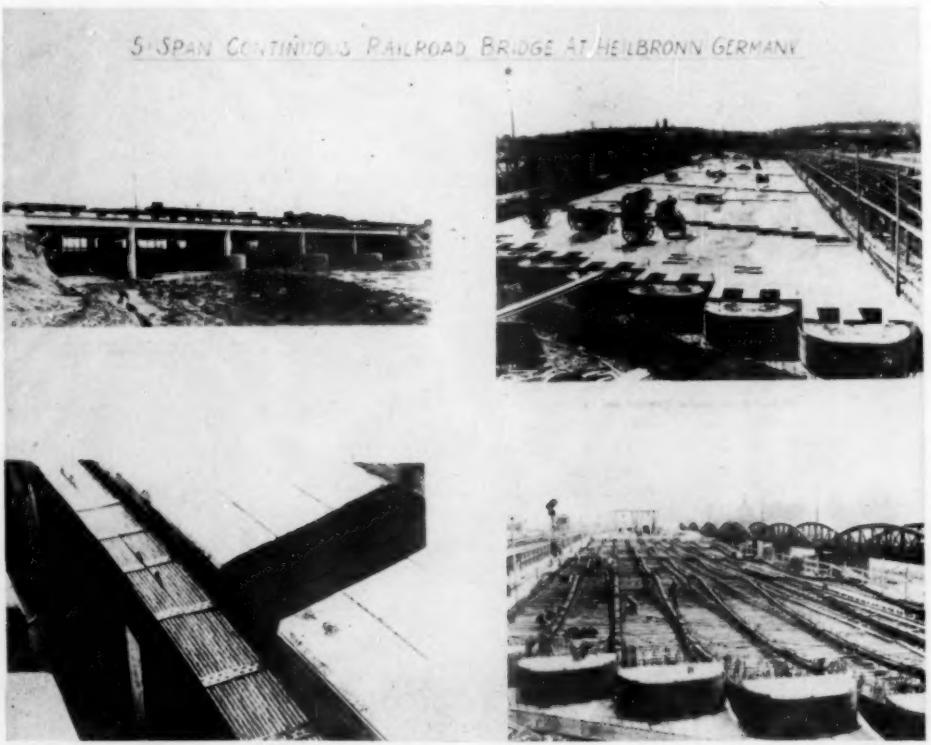
Fig. 5

of Walnut Lane was $21.50 per sq. ft. for the deck, erected. The estimate cost for Cathedral Road is only $13.60 per sq. ft., or 30% less.

Fig. 2 shows the falsework for a bridge with multiple spans of 148 ft. by 27 ft. wide. Two pairs of 36 inch wide flange girders carry the falsework between piers with pipe struts from the pier footings to the third points of the span. This arrangement is necessary because the water depth is 90 ft. and the deck is 50 ft. above water level. A timber platform is erected on the falsework carrying the formwork for the concrete cross-section.

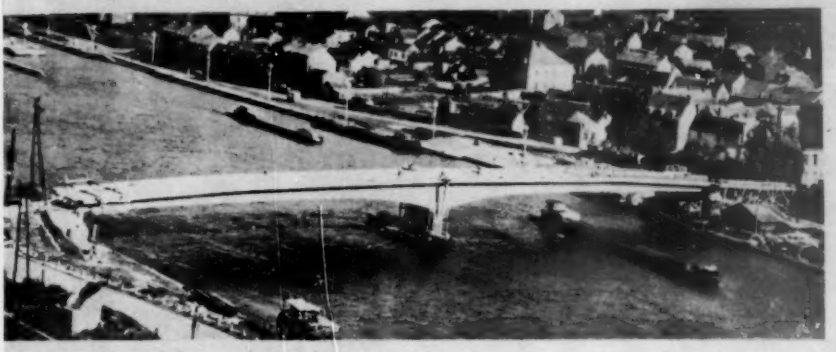
Fig. 3 shows an arrangement for the same

Fig. 6

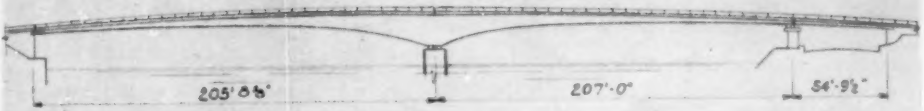


SCLAYN - BELGIUM

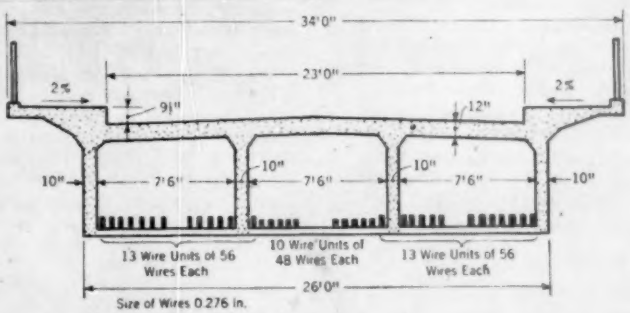
CAST IN PLACE CONTINUOUS 2 SPAN (205', 207') HIGHWAY BRIDGE PRESTRESSED WITH INDIVIDUAL CABLES OF PARALLEL WIRES



GENERAL VIEW



ELEVATION



TRANSVERSE SECTION

Fig. 7

span using two exterior trusses spanning the full length carried on sand jacks which in turn rest on temporary corbels on the piers. In this case the drop of the sand jacks must be equal to the depth of the girder members so that the falsework can be dropped clear of the concrete section before moving horizontally. The exterior of the formwork is sheeted so that it may be moved by floating.

Fig. 4 shows a more elaborate arrangement of self-centering falsework. The principles are the same as shown in Fig. 3 but in order to use I-section webs the outside forms are hinged to clear the bottom flange and separate filler-boxes are used on the inside of the web. This falsework is being used in the construction of the Porto Alegre bridge near Sao Paulo, Brazil, which is 6 kilometers in length composed of multiple 120 ft. spans continuous over three spans and simply-supported spans 74 ft. in length for approaches. The bridge is 34 ft. 2 in. wide. The self-centering falsework requires 23 lbs. of structural steel per sq. ft. of deck.

Fig. 5 shows a suspended system of falsework developed for a bridge with three spans 225 ft. – 320 ft. – 225 ft. In this case the spans are too long for self-centering falsework and the elevation of the deck too high above the bottom of the river to consider the use of temporary bents. Temporary towers are erected on the piers from which a platform is hung from sloping suspenders. The formwork is erected on this platform as shown in the cross-section with the suspenders passing through holes in the

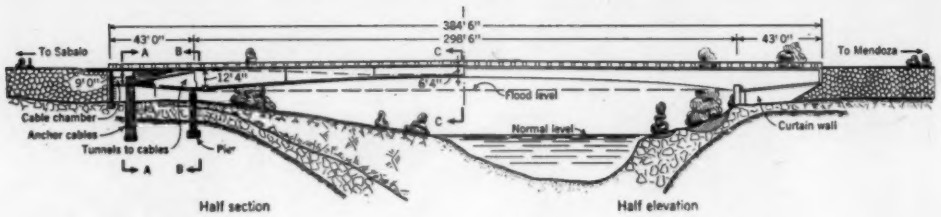


Fig. 8—Typical design details of several bridges in Cuba using exposed unbonded prestressing tendons.

deck on the inside of the web. For this type of falsework it is necessary to have adequate adjustment in the suspenders to correct for deflections as concrete is placed.

Fig. 6 shows construction details for a five span continuous flat slab railway bridge designed by Leonhardt in Germany. The spans are 69 ft. x 60 ft. – 60 ft. – 72 ft. – 64 ft.

The lower righthand corner shows the prestress casings carried on chairs resting on the formwork erected for the whole length of the bridge. Semi-circular end-blocks are used at each end of the bridge which rests on runners for subsequent movement.

As shown in the bottom lefthand corner the prestress casings are filled with multiple layers of 7-wire strands with spacers to keep them relatively positioned both vertically and horizontally. The strands are placed by means of a tractor which carries a loop of strand from one end of the bridge and deposits it over the opposite end-block and repeats the operation on the return journey. When the boxes are filled they are closed with waterproof sectional cover.

The upper righthand corner shows the placing of concrete with two cavities left behind each end-block for jacks. Open construction joints are provided over the piers to be filled after the formwork has taken the deflection from the added weight of the concrete. When the concrete has attained its required strength, jacks are inserted in pairs behind each end-block and are connected in series to a power-driven pump which jacks out the end-blocks simultaneously at both ends of the bridge until the required stress is reached in the strands. This is cross-checked by measuring the elongation through windows left in the casings at various points throughout the length of the bridge. When the end-blocks have reached their final movement concrete is placed between the block, and end of the slab and jacks are removed. The jack cavities are then filled with concrete which also surround the end-blocks which become part of the slab, and the prestress casings are pressure-grouted. This system of construction has been widely used for long span bridges in many countries.

Fig. 7 shows a different arrangement for a bridge of two continuous 205 ft. spans designed by Professor Magnel in Belgium. A box-section with four webs is used to carry the 34 ft. wide deck. Prestressing units are not built into the webs but are left exposed within the boxes. A deep haunch is used at the center pier so that even with straight

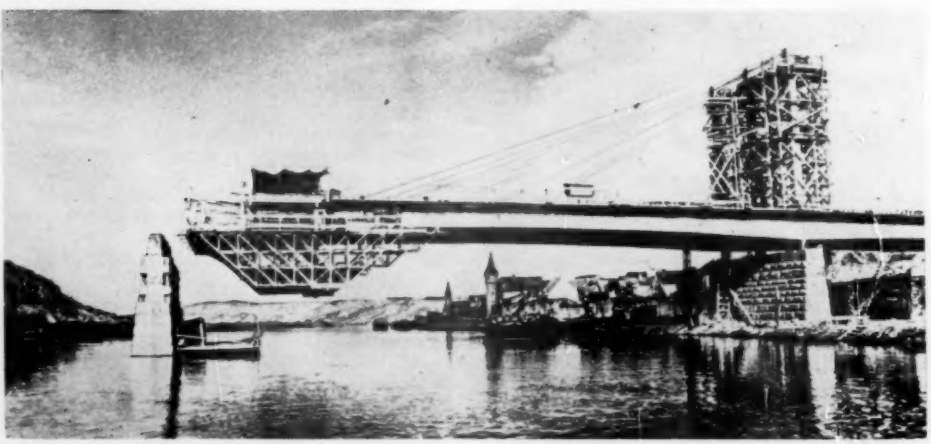


Fig. 9—Bridge over River Main at Karlstadt built in 1952—Dywidag Prestressed Concrete—temporary steel cable supports and progressive formwork.

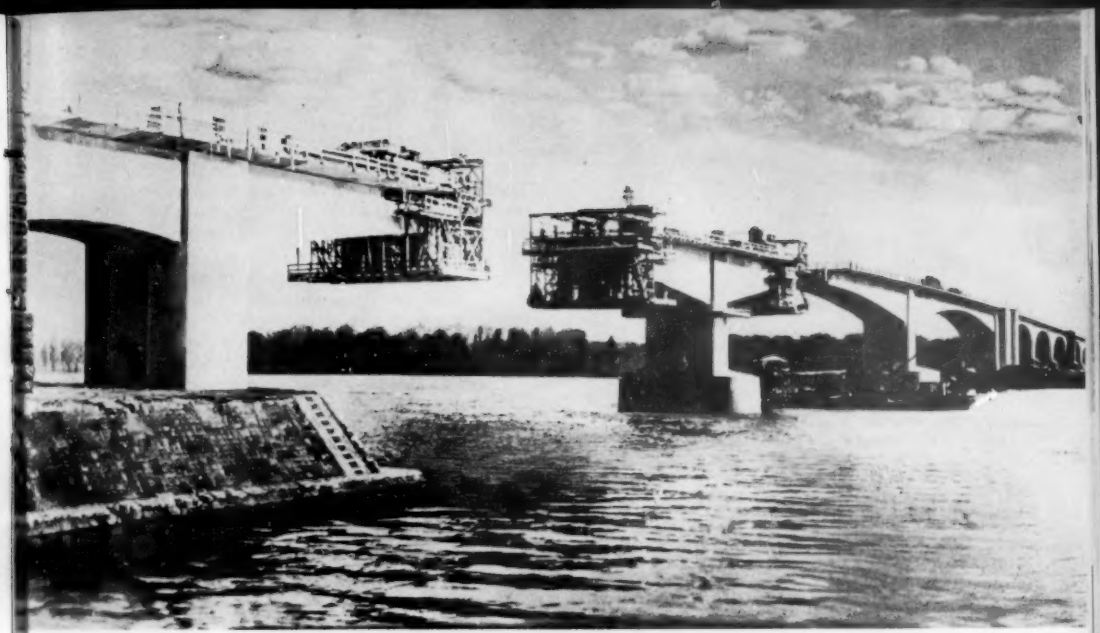


Fig. 10A—Nibelungen-Bridge at Worms—progressive formwork.

cables continuity is developed over the supports.

Fig. 8 shows details of similar construction which has been used for several large span bridges in Cuba designed by Prof. Luis Saenz. In this case a double-box section is used to carry the 20 ft. wide deck with haunches at supports to develop continuity with reduced deflections of prestressing cables which are unbonded and exposed within the boxes. Roebling cables and end anchorages have been used in these Cuban bridges.

Fig. 9 shows a system of progressive cantilever construction developed by Dr. Ulrich Finsterwalder of Dyckerhoff & Widmann of Munich, Germany. This particular bridge has four spans of 130 ft. each. A temporary tower is built over the pier with suspenders extending out to the deck. The cross-section of the bridge is poured in increments of 30 ft. in cantilevered formwork and is prestressed to previous sections by use of high strength rods before the cantilever formwork is advanced for the next pour. With the use of high early strength cement Dr. Finsterwalder claims that their average rate of production is about one form-length of 30 ft. per week for this kind of construction. Many beautiful large span bridges have been built in Germany by this method. This type of construction is probably too costly to use where self-centering falsework or temporary bents are suitable, but it is probably competitive with suspension systems for high level spans.

Figs 10A and 10B show another use of the progressive cantilever type of construction for the well-known bridge at Worms, Germany, with three continuous spans of 335 ft. – 375 ft. – 333 ft. In this case the temporary suspension system was unnecessary as the forms were advanced equally on each side of the piers balancing out the construction loads. Careful allowance for temperature changes, humidity and deflections must

Fig. 10B—Nibelungen-Bridge at Worms—Dywidag Prestressed Concrete progressive cantilever construction—under view showing detail of center hinge.

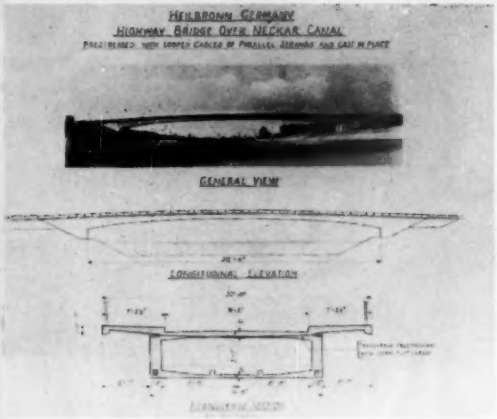


Fig. 11

be made in setting the forms to insure that the deck surface will match up when the forms meet at mid-span.

I would now like to show a number of illustrations of bridges built or designed by the different methods discussed.

Fig. 11 is the Neckar Canal bridge in Germany designed by Leonhardt. It has a center span of 312 ft. 6 in. with flanking spans each side of 62 ft. which are camouflaged with curtain walls to give the appearance of an abutment. A single box girder section is used with wide cantilever overhangs to carry the total width of 30 ft. 10 in. Only one prestressing cable is used in each web to carry this long span. Transverse prestressing is used in the deck to develop the overhangs. At mid-span the depth span ratio of this bridge is 1/61 which gives a very slender graceful appearance.

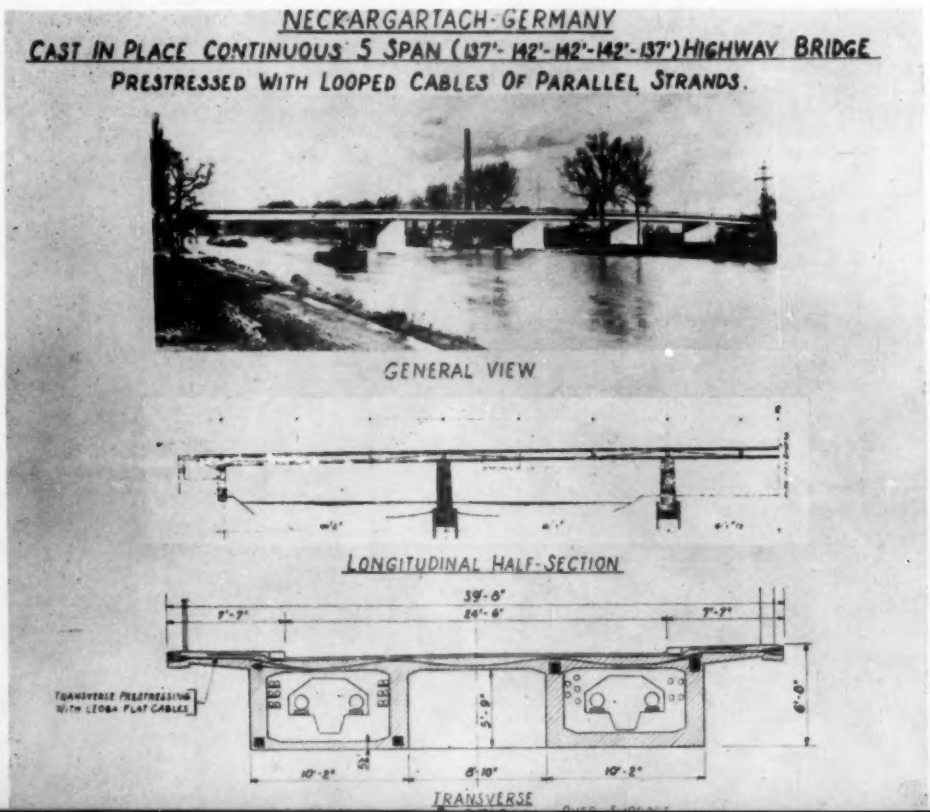
Fig. 12 shows a bridge continuous over five spans of approximately 140 ft. each using parallel cords throughout. Two box girders cast monolithically with the deck are used in this case with four prestressing cables carried in the webs. The entire area of the bridge was formed up with temporary falsework so that it could be cast in one continuous monolithic pour and prestressed in a single operation. I think you will agree it is a satisfactory type of design with a nice clean appearance which would be suitable for many of our expressway bridges.

The quantities of material per square of deck required for this bridge were:

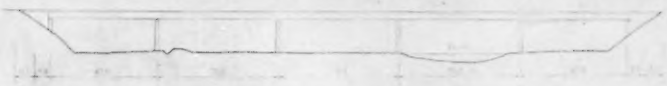
Concrete	.061 cu. yds.
Prestressing Steel	5.53 pounds
Mild Steel	8.4 pounds

Fig. 13 shows a more spectacular prestressed bridge over the Danube Valley with five continuous spans. The three center spans are 230 ft. and the end spans 203 ft., also designed by Leonhardt. I made two visits to this bridge during construction and

Fig. 12

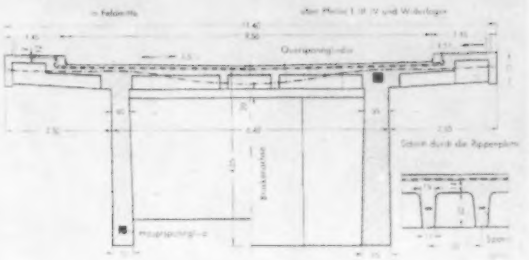


GENERAL VIEW



LONGITUDINAL ELEVATION

VIEW SHOWING MAIN GIRDER DIAPHRAGM AND RIBBED SLAB



TRANSVERSE SECTION

Fig. 13—Continuous Prestressed Concrete bridge over Danube Valley at Untermachtal, Germany; five spans: 203′ — 230′ — 230′ — 230′ — 203′.

was fortunate to be there on the day when the prestressing was done. Because it is comparatively narrow the entire width was cast monolithically. A gridded deck as shown in the bottom righthand corner was used to reduce dead weight with transverse prestressing being used in each transverse web to develop the negative moment over the supporting girders.

The quantities of material per square of deck required for this bridge were:

Concrete	.084 cu. yds.
Prestressing Steel	6.81 pounds
Mild Steel	8.2 pounds

Fig. 14 shows the Rio Cuyaguateje bridge in Cuba designed by Professor Saenz for the Pan American Highway at the western end of the island. It has a center span of 298 ft. with flanking spans of 43 ft. at each end. A good description of this bridge is given in the December 1955 issue of Civil Engineering.

Fig. 15 shows a fine example of a completed bridge design by Dr. Finsterwalder. It is the Worms bridge discussed earlier in the description of his method of construction.

Fig. 16. Movable spans are frequently required in these long multiple span bridges which often works to the disadvantage of prestressing. The movable spans are always designed in structural steel. When they know they are competing with prestressed concrete for the fixed span the steel companies have a tendency to raise their price of the movable span and take the difference off the cost of the fixed span, or give the contractors quoting only on the prestressed design such a high cost for the movable span alone that it puts prestressing out of the running. This problem was overcome at the

Fig. 14

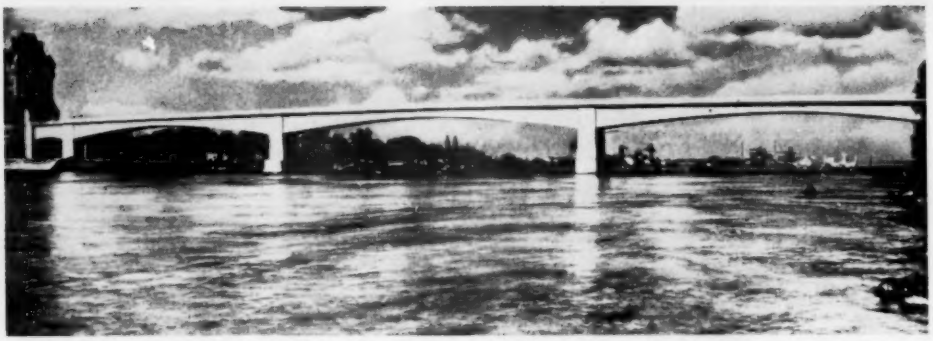


Fig. 15—Road bridge across the Rhine at Worms—cantilever construction in Prestressed Concrete, Dywidag System; spans: 335' — 375' — 333'

Porto Alegre bridge in Brazil by designing a lift span in prestressed concrete. Fig. 16 shows a rendering of this span which is 131 ft. long by 34 ft. wide. Both the towers and span are designed in prestressed concrete. The weight of the prestressed concrete span is about 50% greater than a corresponding span in structural steel which requires increasing the size of the cables and counterweights proportionately.

Fig. 17 shows details of a design of a typical expressway overpass developed by The Preload Company for State of Massachusetts. A flat slab continuous over four spans and cored to reduce weight is carried on circular columns. In order to eliminate an exterior cap over the columns heavy prestressing is used transversely in the deck over columns to develop the punching shear. The slab thickness is only 22 inches, giving a depth span ratio of 1/32. While this type of design requires more prestressing steel than what would be needed for a deeper section the added cost is often more than offset by the reduction in the volume of cut and fill required to maintain the sight lines. A similar bridge in structural steel required an over-all depth of 42 inches. The extra 20 inches of depth called for an additional 6000 cubic yards of cut and fill to maintain the same sight lines.

A more economical type of expressway overpass in which the center pier is eliminated, results in a main span of 120 ft. with flanking spans of 40 feet. Five webs are used in the cross section with a closed soffit. The prestressing is carried in the webs and is continuous over supports. Bond is developed by pressure grouting after prestressing.

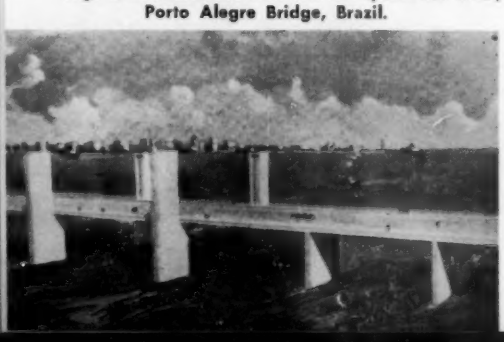
The quantities of material and unit cost per sq. ft. of deck required in this design are:

Concrete	0.063	cu. yds.	@	$60.00 =	$3.78
Prestressing Steel	3.3	lbs.	@	0.70 =	2.21
Mild Steel	7	lbs.	@	0.12 =	.84
TOTAL unit cost of deck					$6.83

Where a center pier is used quantities and unit costs of materials per sq. ft. of deck for a corresponding steel bridge would be:

Structural Steel	26	lbs.	@	$ 0.18 =	$4.68
Concrete	.022	cu. yds.	@	50.00 =	1.10
Mild Steel	4.5	lbs.	@	0.12 =	.52
TOTAL unit cost of deck					$6.30

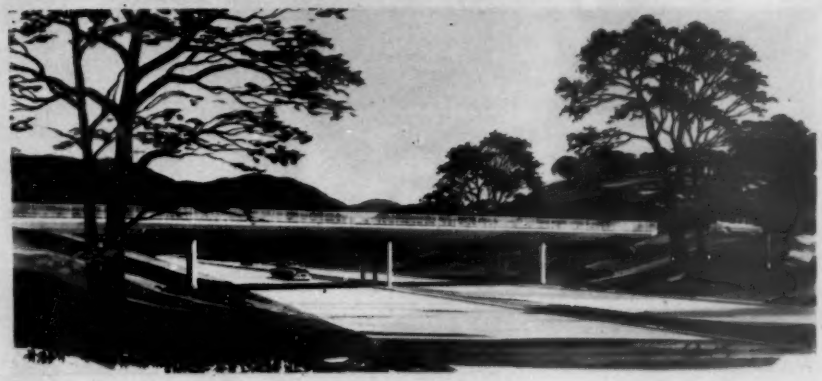
Fig. 16—Prestressed Concrete lift span—131'x43', Porto Alegre Bridge, Brazil.



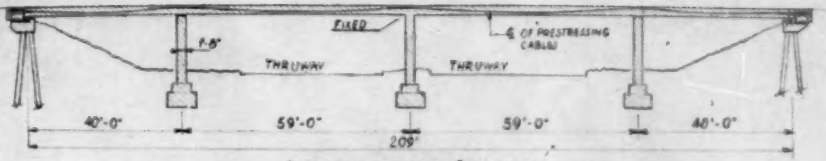
The cost of the center pier would just about offset the extra $0.53 per sq. ft. for the prestressed bridge which would be free of maintenance and eliminate the accident-hazard of the center pier. I have not seen a similar design for a steel bridge without a center pier, but this would add about 24 lbs. of structural steel per square foot or about $4.32 per sq. ft. in cost.

Fig. 18 shows a bridge designed by The Preload Company for the Corps of Engineers crossing the Potomac Canal at the new Little

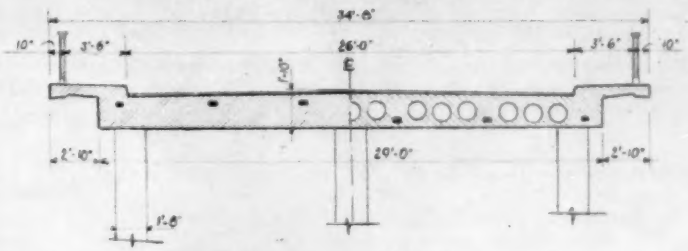
CONTINUOUS 4 SPAN (40'-59'-59'-40') HOLLOW SLAB PRESTRESSED WITH LOOPED CABLES OF PARALLEL STRANDS.



GENERAL VIEW



LONGITUDINAL SECTION



OVER SUPPORT AT MID-SPAN

TRANSVERSE SECTION

Fig. 17

Falls Pumping Station west of Washington. While the main span of 216 ft. is insignificant compared with some of the European bridges discussed earlier, I believe this is the longest prestressed concrete span yet to be placed under contract in the United States. The design is somewhat similar to the 315 ft. Neckar Canal bridge using a single box girder section with one large prestressing casing in each web and overhanging spans at each end to develop the negative moment over supports. The girder depth at mid-span is 7 ft. which gives a depth span ratio of 1/31.

An imposing prestressed concrete bridge has recently been designed across the Potomac River in the heart of Washington. With the concurrence of the District of Columbia a co-venture was formed between the Freyssinet and Preload companies to undertake this design and a toss of a coin decided that the name of the co-venture would be "Freyssinet-Preload." Moore & Hutchins of New York are associate architects. Two contracts were let by the District of Columbia to prepare preliminary designs and renderings for this bridge which lies just north of the Memorial Bridge in view of the Washington Monument and the Lincoln Memorial. One contract is for a prestressed concrete design and the other contract for a structural steel design. Certain requirements were laid down governing the architecture of the bridge. Spans must not be less than 160 feet. Parallel cords must be used so that the bridge would not compete in appearance with the multiple-arch Memorial Bridge. Piers had to be stone-faced.

Fig. 19 shows an elevation rendering of this prestressed bridge. It has three 184 ft. spans on the Virginia side, shown on the left; an elevated filled section over Roosevelt

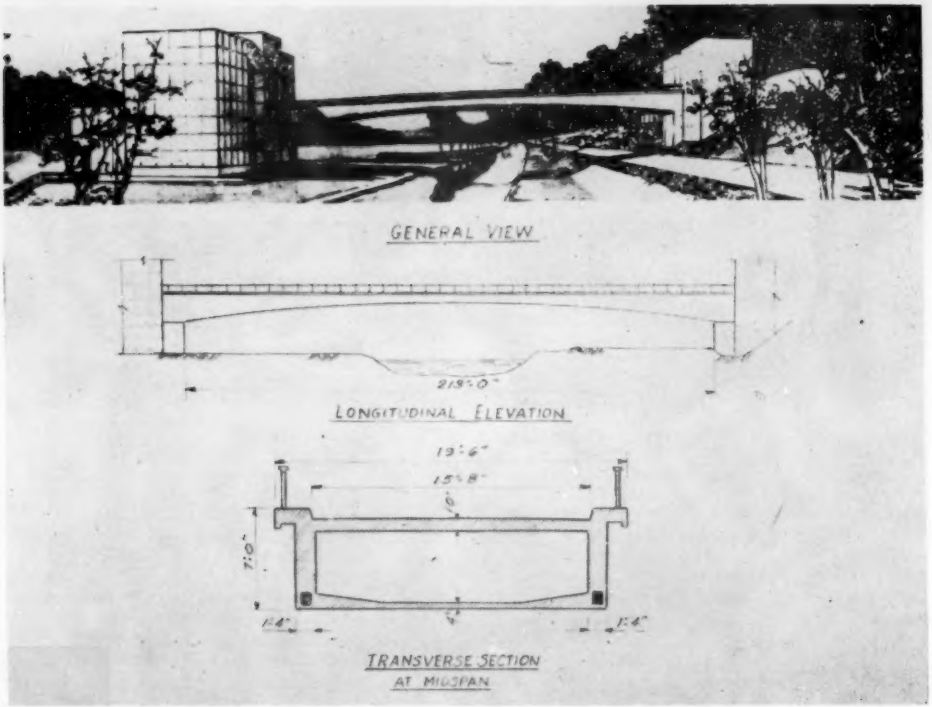


Fig. 18—Little Falls Bridge, Potomac Canal, Washington, D. C.—Prestressed Concrete continuous cast-in-place bridge—single span 216'—girder depth mid-span, 7' 0"—depth span ratio 1/31.

Island 475 ft. in length, and nine 184 ft. spans on the Washington side shown on the right.

Fig. 20 shows a cross-section of this bridge. The bridge has a width of 92 ft. carrying three lanes of traffic in each direction. It has a 5 ft. sidewalk each side and 4 ft. media strip. The bridge is designed for continuity over groups of three spans with two webs spaced 29.6 ft. apart in each half section and cantilever overhangs of 10 ft. on each side. The deck is designed as a waffle-slab to reduce dead weight. This general arrangement will permit four re-uses of falsework which will be the floating self-centering type described earlier.

The architecture of this bridge with the distinctive treatment of piers and wide overhangs was approved with favorable comment by the Fine Arts Commission of the District of Columbia.

The quantities of materials for the superstructure per sq. ft. are:

Concrete	.065	cu. yds.
Prestressing Steel	4.6	pounds
Mild Steel Reinforcing	11	pounds

The engineers' estimate for this design, subsequently checked and confirmed by several large contracting companies, is $4,751,000. The unit cost per sq. ft. of deck for the various elements are:

Superstructure	$13.10
Substructure including stone-facing of piers	$ 8.04
Total Bridge Structure	$21.14
Elevated fill section over Roosevelt Island	$18.60

The engineers' estimate for a corresponding bridge in structural steel was $7,900,000 or approximately 40% higher to which must be added the capitalized cost of the maintenance for painting estimated at $14,000 per year.

Fig. 19

This job is as good an example of the important saving which can be realized with cast-in-place prestressed concrete design for long multi-span bridges.

I might cite just one more example of a bridge we are now designing for five 120 ft. spans 68 ft. wide on a 1700 radius curve. Competitive bids are to be taken for prestressed concrete and structural steel designs. The prestressed concrete design calls for the following superstructure quantities per sq. ft. to which are applied conservative unit cost:

Concrete	0.64	cu. yds.	@	$80.00 =	$5.12
Prestressing Steel	3.4	lbs.	@	0.55 =	1.87
Mild Steel	7	lbs.	@	0.12 =	.84
TOTAL unit cost of superstructure					$7.83

The corresponding quantities for the superstructure per sq. ft. of the structural steel design with unit costs applied are:

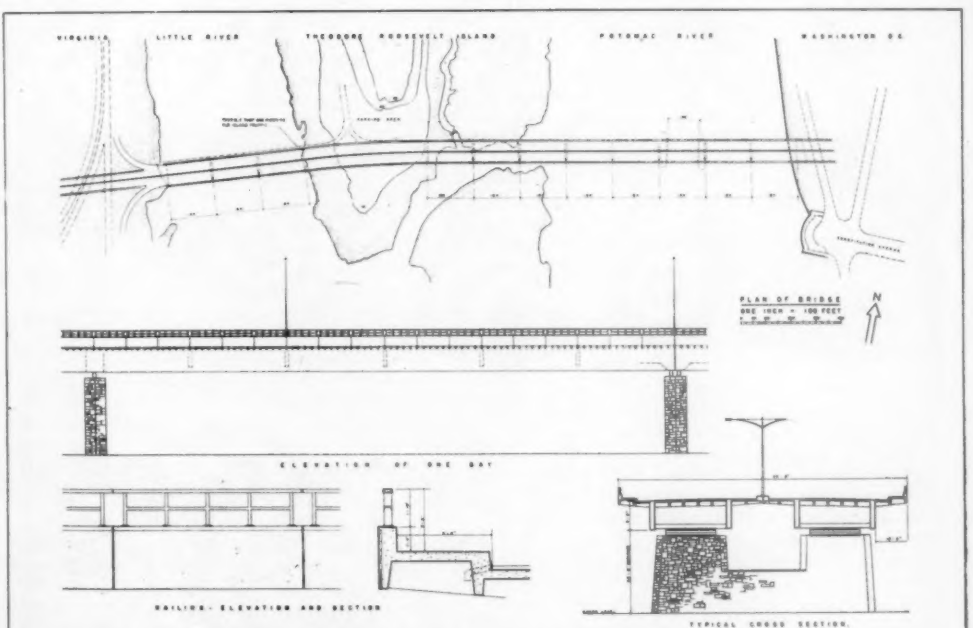
Structural Steel	52	lbs.	@	$ 0.20 =	$10.40
Concrete	0.22	cu. yds.	@	60.00 =	1.32
Reinforcing	4	lbs.	@	0.12 =	.48
TOTAL unit cost of superstructure					$12.20

Looking at it another way, the structural steel would have to be erected for 11.6c per lb. for the cost of both bridges to be equal. I think you will agree this would be impossible at today's steel prices for a bridge built on a curve.

Savings of 30-50% compared with structural steel are not unrealistic for structures of this kind. This compares with savings of 2-18% shown in competitive bidding against structural steel and reinforced concrete for the short span precast prestressed bridges constructed to date by the State Roads Department of Florida.

Fig. 21 shows a cross-section of the Montevideo stadium covering a circular building 308 ft. in diameter. At the outside wall the cable terminated in a large horizontal compression ring carried on top of the wall and at the inside terminates in a structural steel tension ring 19 ft. in diameter. Trapezoidal precast concrete panels 2 inches thick are hung on these cables by means of projecting reinforcing hooks at the four corners as

Fig. 20



PRESTRESSED CONCRETE DESIGN - CAST IN PLACE CONSTRUCTION, FOR THE PROPOSED CONSTITUTION AVENUE BRIDGE OVER THE POTOMAC RIVER, WASHINGTON, D. C.

FREYSSINET - PRELOAD, CONSULTING ENGINEERS. NEW YORK, N.Y.
MOORE AND HUTCHINS, ARCHITECTS

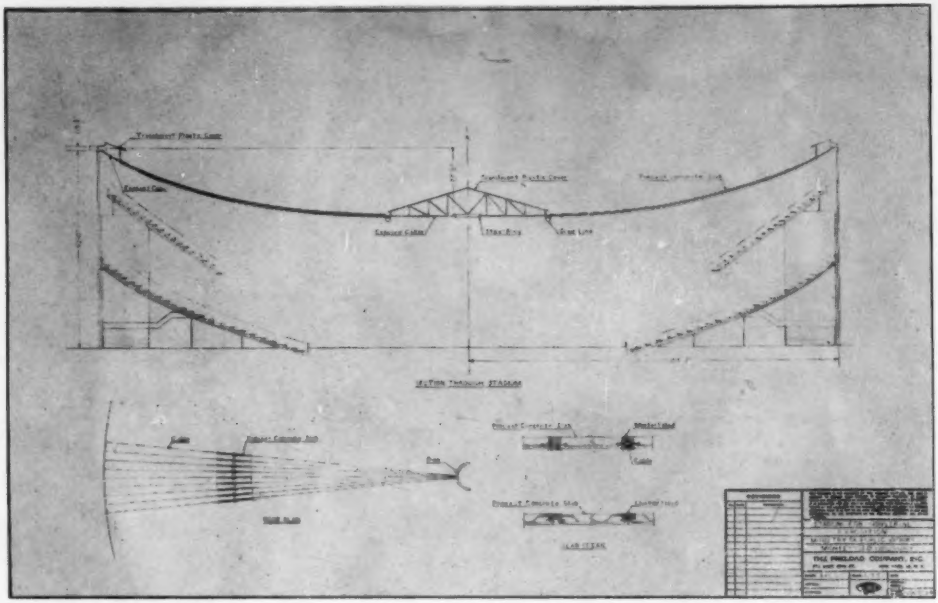


Fig. 21

shown on the small cross-section. In order to provide some prestressing in this roof system to insure tight joints and give additional stiffness for wind loads, Mr. Viera came up with a brilliant but simple suggestion for prestressing the roof. After the panels were all set in place each panel was loaded with concrete block equivalent in weight to the maximum live load which elongated the cables and expanded the joints between panels. In this condition the joints were grouted and when the grout had attained sufficient strength the superimposed dead weight was removed leaving a prestress in the cables and a compression in the slab and joints. In order to avoid the high bending moments which occurred at the supporting edges of such systems the concrete panels were terminated 5 ft. from the outer ring and the space covered with an articulated plastic housing. The center of the roof is covered with a light steel truss with a translucent plastic covering to provide natural flood lighting to the interior.

Fig. 22 shows a rendering of the exterior of the sports arena which is now nearing completion, and also an interior view of the stadium filled to capacity with 24,000 happy spectators showing little concern in the fact that they are under a prestressed concrete roof, 2 inches thick, spanning 308 ft.

Such a roof system cannot, of course, be used in a country which is ever likely to have snow, but we are developing this general concept of prestressing suspended roofs for large diameter tank and other structures where a central tower can be used carrying the cables at the center at a higher elevation than the outside wall to provide drainage to outside. This type of prestressed suspended construction because of its very low cost provides the most economical method of covering large areas without any intermediate supports, or just a single central column.

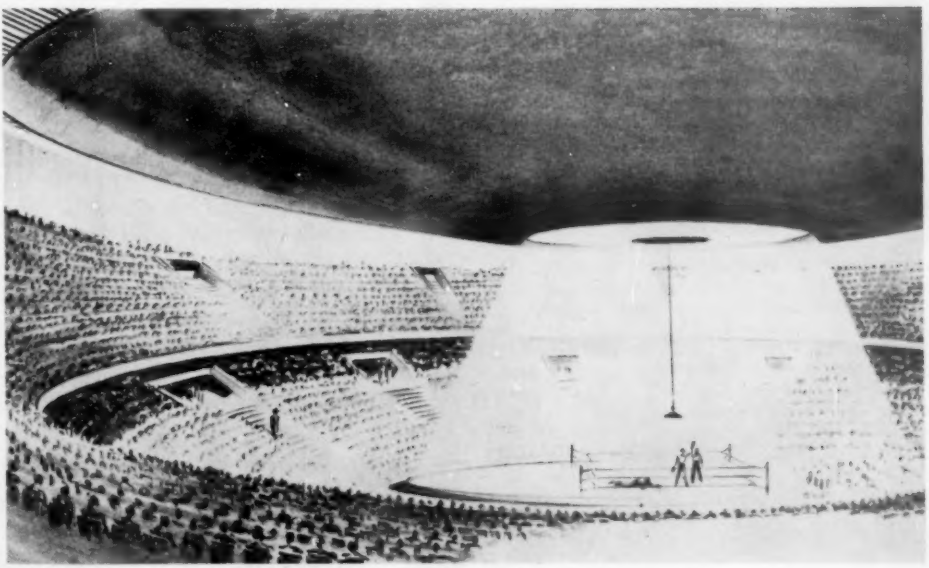
CONCLUSION

Now, gentlemen, I don't want to detract in any way from the fine work which we have done in developing and standardizing our design procedures and construction method for prestressed concrete for relatively short spans lending themselves to precast construction, but I hope I have given you enough data to convince you that our big job in the future is to develop prestressed concrete for long span heavily-loaded structures where it will show much greater saving in costs compared with older methods of construction than our work heretofore might indicate.

Convincing the people who have to pay for these structures of what we know to be true will not be an easy task. In competing with structural steel we are up against a wealthy and highly-organized industry which does not hesitate to discredit prestressed concrete wherever possible and drastically cut prices on individual jobs where prestressed concrete may offer a competitive threat. For these larger structures we cannot afford to give the owners free designs merely for the privilege of competing with other methods of construction for which an ample fee for design has been paid which does not have to be included in the construction price.

Some highway and bridge authorities are precluded by law from giving out consulting contracts for design of structures and yet

Fig. 22—Stadium for National Industries Exhibition, Montevideo, Uruguay—310′ diameter stadium, 83′ high, with suspended roof system.



they do not have the trained personnel to make efficient prestressed design for these larger structures. Other authorities follow the traditional pattern of awarding omnibus consulting contracts to the large general consulting firms for large sections of highway including structures at comparatively low fees. These firms, even when they admit that prestressed designs would save in construction costs many times the added cost of engineering, simply cannot afford at the low over-all fee to subcontract the design of major bridges to specialists.

Even when we convince owners that large savings can be made with p/c they often feel it necessary to insure their own judgment by insisting that alternate designs be made and bids taken for prestressing and some older method of construction only to find that their engineering budget will not stand the cost of making two complete sets of alternate designs and specifications.

In order to overcome these obstacles we must have patience in the knowledge that big things never come easily. We must refrain from recommending p/c where its advantages are marginal and concentrate on selective jobs where we must convince highway and bridges authorities that to obtain the full benefits of prestressing they must be prepared to exclude major structures from general consulting contracts and award these separately to specialists in this field at adequate fees to cover the higher cost of engineering involved.

When we have successfully done this on a few major structures, such as I have described, the inertia and resistance will evaporate and owners and their financial underwriters will hasten to take advantage of the large savings in cost and maintenance which we have to offer.

Unfortunately, we do not yet have a well-financed national organization, as do the older methods of construction, to promote the interest of prestressed concrete and fight our battles in the lobbys of Washington and the state capitols. I would like to conclude with this thought – why don't you expand the Prestressed Concrete Institute into a truly national organization with headquarters in Washington and officers chosen from all parts of the country representing the interests of the consultants, contractors, material suppliers and manufacturers. To give it national stature this Institute should seek to represent the United States in the Federation Internationale de la Precontrainte and should join with A S C E and A C I in sponsoring the Recommended Practice for Prestressed Concrete discussed in Mr. Germundsson's paper. It should also have authority, under its by-laws, to prevent its members from indulging in unethical practices, such as making exaggerated claims for any particular proprietary methods, blue-sky promotion of prestressing where it is not justified and unfair competitive practices amongst its members. We need such a national organization and I think our industry has now reached the size where we can support it financially.

If we do these things, there is no reason in the world why we should not be building as high a proportion to the total of prestressed concrete bridges in this country as they are doing in Europe, but long before we attain this goal prestressing will have become a large and important industry in this country.

In its report to the President published in 1952, the President's Materials Policy Commission, under the chairmanship of William S. Paley, recommended the wider use of prestressed concrete as a means of conserving our national resources to meet our expanding economy. It is up to all of us to get behind this recommendation and push prestressed concrete over the top for both large and small structures.

REFERENCES

DEAN, W. E., "Comparative Costs of Prestressed Construction and Conventional Types in Trestle Spans of Florida Bridges," Highway Research Board, January 1956.

DOBELL, C., "Prestressed Concrete in Highway Bridges and Pavements," Highway Research Board Proceedings, January 1953.

DOBELL, C., and LEONHARDT, F., "Wrap-Around Cables Prestress . . . Long-Continuous Girder Bridge," Engineering News Record, September 3, 1953.

GUYON, Y., "Prestressed Concrete," Contractors Record and Municipal Engineering, London, 1953.

LEONHARDT, F., "SPANNBETON Fur Die Praxis," Verlag Von Wilhelm Ernst & Sohn, Berlin, 1955.

LIN, T. Y., "Design of Prestressed Concrete Structures," John Wiley & Sons, Inc., New York, 1955.

MAGNEL, G., "Prestressed Concrete," Second Edition, Concrete Publications, Ltd., London, 1950.

SAENZ, L., and MARTIN, I., "Prestressed Concrete Bridge in Cuba Spans Nearly 300 Ft.," Civil Engineering, Dec. 1955.

SCHOFIELD, E. R., "Construction Starts on Prestressed Concrete Bridge in Philadelphia," Civil Engineering, July 1949.

WALLEY, F., "Prestressed Concrete Design and Construction," Published for Ministry of Works by Her Majesty's Stationery Office, London, 1953.

eliminate field lubrication

Exhaustive field tests have proved Whiteley graphited expansion plates give anti-friction performance with an absolute minimum of field service, even under the most adverse conditions. These dependable, self-lubricating bronze plates are approved by construction engineers for floating bearing linkage on heat exchangers, boiler vessels, and bridges.

SPECIAL PROCESS

The Whiteley graphiting process inserts graphite into the plate under hydraulic pressure, producing permanent lubrication — these graphited plates will never be affected by moisture or temperature. The Whiteley plate is permanently lubricated when manufactured. Therefore, no additional lubrication at the time of field installation is necessary.

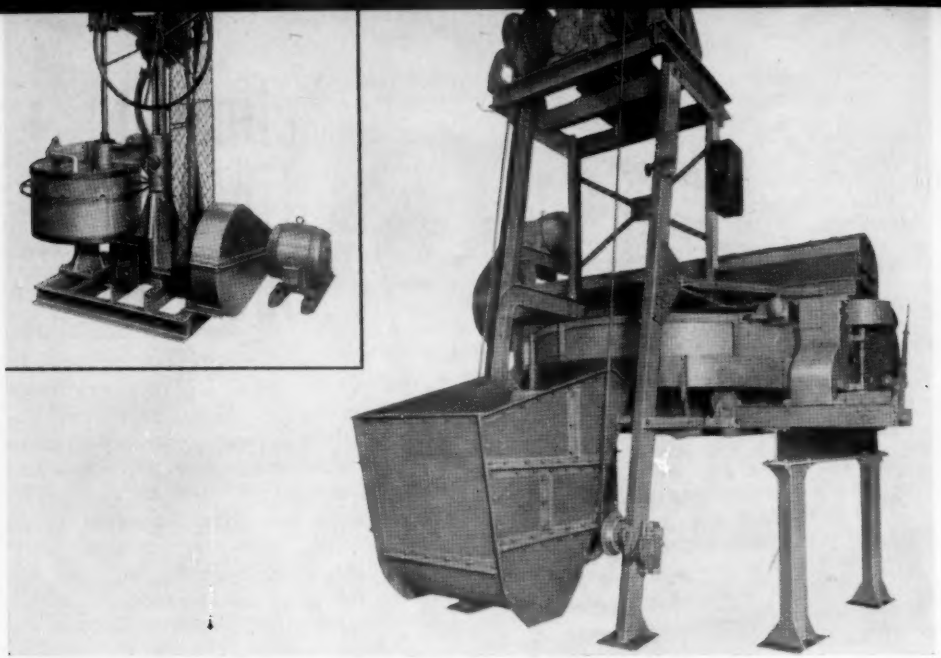
Self-lubricating expansion plates available in thicknesses and sizes according to your specifications and needs. Supplied with graphite lubrication on one or both sides.

Additional information and test reports available on request.

THE
Whiteley Bearing
CORP.

1238 S. Ashland Avenue
Chicago 8, Illinois

• Other graphited bronze products that bear the WHITELEY name are Thrust Washers, Bearings — standard loop grooved or the drilled hole type— and Bronze Bushings.



CUMFLOW mixers are available in 6 sizes, from 2 cu. ft. lab. models to 50 cu. ft. production models. Stationary or portable, with or without skip. Power optional. Mixers furnished for electric, gasoline or Diesel power operation.

CUMFLOW SCIENTIFIC MIXING SYSTEMS

Manufactured by THE LINER CONCRETE MACHINERY COMPANY, LTD., of Gateshead, England

The CUMFLOW SCIENTIFIC MIXING SYSTEM is designed and built expressly for the precast and prestressed concrete industries. Using this system you can obtain from 15 to 40 uniformly mixed batches of concrete per hour. CUMFLOW mixes any type concrete desired – slump, plastic or wet mix, lean or rich mix, fine or coarse mix, light or heavy mix –with 100% efficiency and uniformity.

The CUMFLOW SCIENTIFIC MIXING SYSTEM consists of a mixer star supported eccentrically over the mixing pan. The star revolves at a relatively high speed. Blades attached to the star are arranged so as to obtain the maximum number of points of intersection during revolution. Batched materials are fed continuously to the mixing star by the pan's revolutions and are mixed evenly and thoroughly in a matter of minutes. Further mixing action is obtained by fixed side blades. This unique system gives an absolute clean pan after each discharge – no part of the mix is carried over to the next batch.

CHECK THESE IMPORTANT ADVANTAGES

- **Every batch uniform; no variation whatsoever**
- **Greater strength concrete with minimum cement content**
- **Balling or segregation impossible**
- **Self-cleaning pan**
- **Up to 11,700 psi concrete possible**
- **From 10% to 25% savings in cement content**

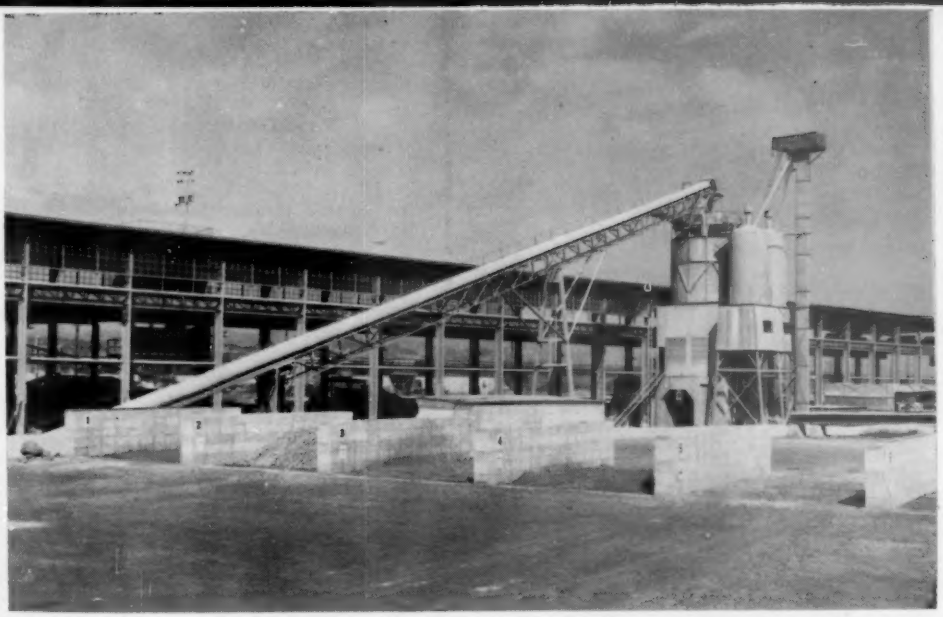
The CUMFLOW SCIENTIFIC MIXER SYSTEM was perfected after years of research to meet a growing demand by engineers for a more intensive mixing element which would give them complete control of the product at all stages . . . a feature not obtainable in conventional-type mixers.

One user of this system, with a cement bill of $1,000,000 a year, says: "On tests, we have found Mixer A gave us 4,200 psi; the CUMFLOW gave us 6,000 psi concrete. We reduced the cement content in the CUMFLOW by 25% and were able to obtain 5,000 psi concrete. We have also obtained concrete of 10,000 to 11,000 psi in this mixer."

The CUMFLOW SCIENTIFIC MIXERS, manufactured by the Liner Concrete Machinery Co., Ltd., Gateshead, England, are built on the lines of a machine tool. They are not to be confused with conventional-type mixers. Available on a world-wide basis, with several parts depots maintained in America for prompt replacement service.

To realize savings of from 10% to 25% on cement costs, and to produce uniform mixes every time, write us, advising in cu. yds., the production needs per hour of your plant. We will promptly forward details of the size mixer best suited to your needs.

The Basalt Rock Co., Inc., Napa, California, is one of the many North American users of LINER CUMFLOW SCIENTIFIC MIXER SYSTEMS.



A few choice dealerships in the United States are still available. Inquiries from qualified firms invited.

Distributor and agent in North America for the LINER CONCRETE MACHINERY CO., LTD., Gateshead, England.

CUMFLOW
MIXER ENGINEERING

9201 SAN LEANDRO STREET • OAKLAND 3, CALIFORNIA
5861 YEW STREET • VANCOUVER 13, BRITISH COLUMBIA • CANADA

The Physical Properties and Methods of Testing PRESTRESSED CONCRETE Wire and Strand

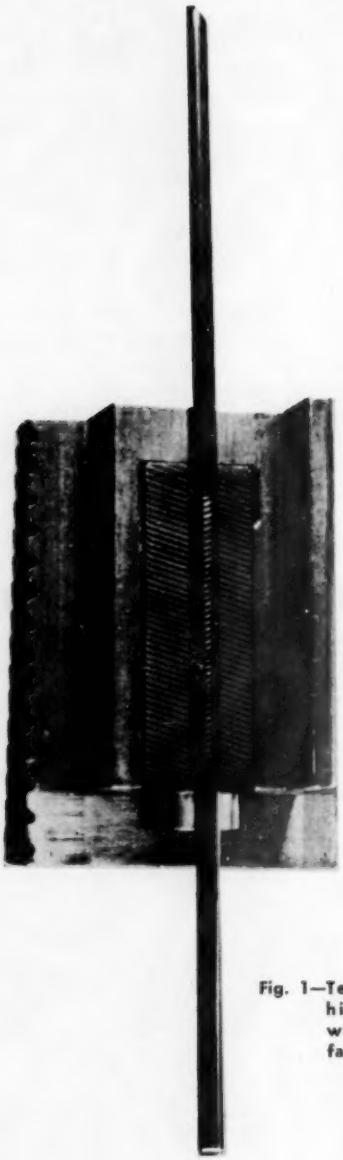


Fig. 1—Tensile test on high strength wire using file face grips.

By H. J. Godfrey
John A. Roebling's Sons Corporation
Trenton, New Jersey

INTRODUCTION

The ACI-ASCE Joint Committee, having recently submitted a preliminary report on "Recommended Practice for Prestressed Concrete," it may be an opportune time to reflect momentarily on some of the physical characteristics of steel wire and strand referred to in the above report, and attempt to visualize their significance. In this discussion of the properties of steel wire and strand we shall describe some of the testing methods by which they are determined and make certain recommendations in regard to specifications.

MATERIALS

This presentation will be limited to the two principal high strength materials used in prestressed concrete – cold drawn steel wire and 7-wire strand. Although the properties of these materials are quite different from those of reinforcing bars used in conventional reinforced concrete, it is of interest to know that all of these materials are made with the same basic elements, namely, carbon, manganese, silicon and iron.

The iron content of steel used in reinforcing bars is approximately 99% and for steel wire is approximately 98%. It is for this reason they have some properties which are very similar. However, a 1% difference in the other ingredients and the methods of manufacture make them quite dissimilar in a great many respects. Of primary importance is the carbon content, which, in prestressed concrete wire, may be 4 to 5 times that in ordinary reinforcing steel.

It is quite important for the fabricator of prestressed concrete structures to recognize these differences so that unnecessary difficulties may be avoided.

PHYSICAL PROPERTIES

1. Tensile Strength and Elongation

The tensile strength of a prestressing element is of primary importance since this value generally determines the design stresses in a prestressed concrete structure. The actual ultimate load of a prestressed concrete beam, however, may be dependent upon other factors, such as design and type of loading.

The testing of a wire of such a high tensile strength requires special type grips to insure a reliable test. An example of suitable grips is illustrated in Fig. 1. These grips are lined with replaceable file face inserts which are ground with a small taper on the leading edge of the insert. The load is picked up gradually by the grip and generally, fracture takes place within the test length between the grips.

The hardness of the high tensile wire is such that even the file hard inserts become worn and have to be replaced periodically.

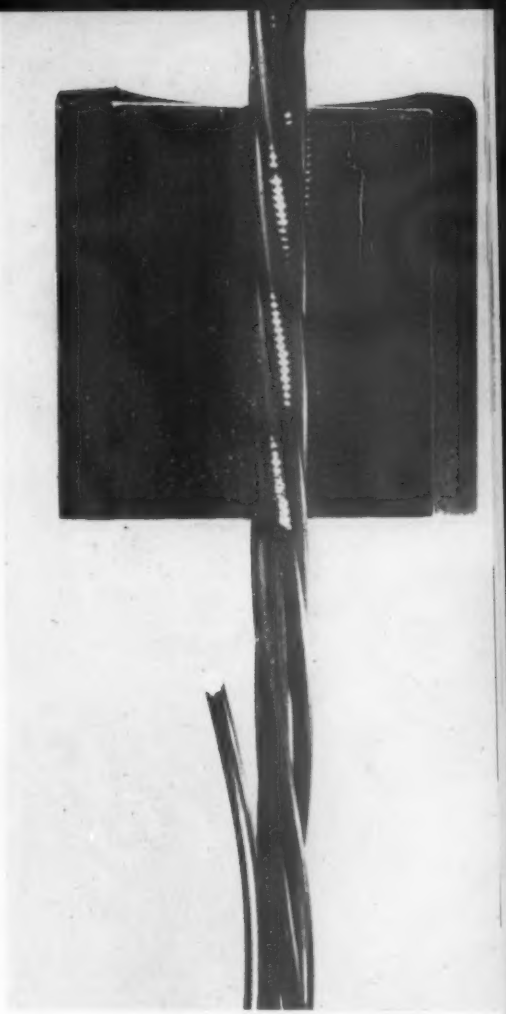


Fig. 2—Tensile test on ⅜" dia. strand using "V" grips—2½" long.

Tensile Tests on ⅜" Dia. Prestressed Concrete Strand
Gage Length = 24 inches

Testing Machine	Grip Length in.	Tensile Strength lbs.	Total Elong. 24" — %	Strand Efficiency %	Type of Fracture
A	2½	20,400	2.4	91.0	One wire at edge of grip. Shear type failure.
A	3¾	21,950	6.7	98.0	All wires at edge of grip. Tensile type failures.
B	4¼	21,800	8.0	97.5	All wires within gage length. Tensile type failures.

TABLE I

The tensile strength of strand is generally expressed in pounds rather than pounds per square inch. For example, the catalog strength of a 3/8" diameter 7-wire strand is 20,000 lbs., which is equivalent to 250,000 pounds per square inch.

A satisfactory test on high strength strand requires a properly designed grip so that the true tensile strength may be obtained. A series of tensile tests were made on a 3/8" diameter strand using serrated "V" type grips which ranged from 2½" to 4¼" in length. The results of these tests are presented in Table I.

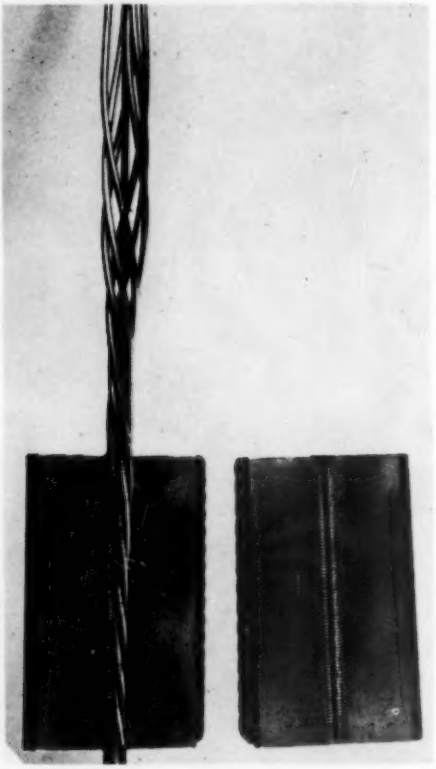


Fig. 4—Tensile test on 3/8" dia. strand using "V" grips—4¼" long.

Fig. 5—Elongation test on strand using 24" gage length.

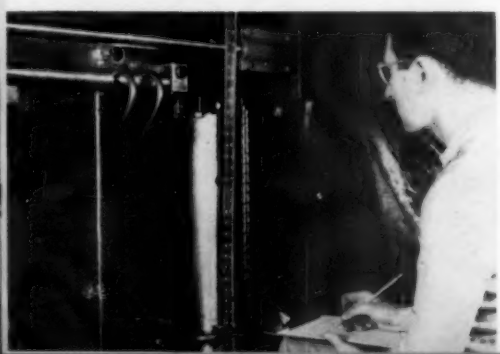


Fig. 3—Tensile test on 3/8" dia. strand using "V" grips—3¾" long.

The data illustrates very clearly that the length of grip used has an important effect on the strength and elongation of the strand.

The tensile strength obtained with short grips might, in some instances, result in a strand being rejected. The low elongation obtained with the 2½" length grips was due to one wire breaking at the edge of the grip with a shear type failure. The shear type failure of the wire was caused by the excessive lateral pressure created by using a short gripping length. As will be demonstrated later, most of the elongation of this particular strand takes place at loads in excess of 20,400 pounds.

It should be noted that although the strand fractured at the edge of the 3¾" length grips, the true strength of the strand was developed and all wires failed with normal cup-cone type tensile fractures. It is therefore not necessary to have the fracture take place away from the grips in order to obtain a good test for tensile strength, but it is essential that the wires break with a normal tensile type fracture.

The elongation of the strand tested with the 3¾" length grip, however, was somewhat lower than the strand which was tested with the 4¼" length grips.

The efficiency of a strand is the ratio of its tensile strength to the summation of the tensile strengths of the individual wires and, as illustrated in Table I, the efficiency was seriously affected by the testing procedure.

Illustrations of the type of grips used, and

resulting fractures of the strands, are shown in Figures 2, 3 and 4.

The method used to measure the elongation of the strand was similar to that described in A. S. T. M. Specifications (A-122-54-T) on Zinc-Coated Steel Wire Strand. A view of the testing procedure is shown in Fig. 5.

The distance between the testing machine grips is approximately 24 inches when the strand in under a load equivalent to 10 per cent of the required tensile strength. The elongation is then determined by measuring the movement of the testing machine head between the 10 per cent load and the actual breaking load. This movement can be read adequately with a steel scale reading to 0.1 inch.

The elongation of a single wire is determined on a 10 inch gage length from gage marks placed on the wire prior to loading. After fracture takes place the two pieces of wire are fitted together and the permanent elongation observed with the use of a steel scale reading to .01 inch. The present tentative specification requires a minimum of 4.0% elongation on prestressed concrete wire.

2. Additional Ductility Indices

Strand having the required tensile strength, elongation and yield strength is considered to be a satisfactory material for prestressed concrete, particularly because the individual wires are tested for strength and ductility prior to fabrication into the strand.

The larger diameter single wires, however, require additional testing to determine whether they have sufficient ductility to insure satisfactory performance.

Ductility is not easily described, and specifying ductility tests to qualify a material for a certain end use is even more complicated.

Specifications may be set up in a tentative form and after experience has shown which type of material has proven to be satisfactory, a final specification can be formulated. Such tests included in the specification can be designed to illustrate the important characteristics of the acceptable material but not necessarily duplicate the conditions under which the material will be used.

We have many types of mechanical tests which are used to measure ductility and in most of these tests the material is subjected to large localized strains which may result in a complete fracture of the material.

A material which can be subjected to the greatest amount of strain without fracturing is considered to be the most ductile. The ability to undergo plastic deformation due to local stress concentrations results in a redistribution of the stress and a safer structure. Although conditions of this sort do not occur frequently, structural elements must be able to perform satisfactorily when subjected to them.

The difficult problem is to determine how much ductility is required and what are the best tests to guarantee that the material will do the job.

Fig. 6—Abnormal tensile fracture of high strength steel wire.

Fig. 7—Wrap test on wire.

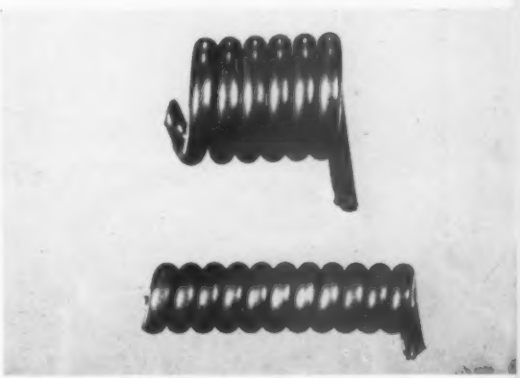


Fig. 8—Wire specimens after wrapping on 3X and 1X diameter mandrels.

The recommendations of the Joint Committee have suggested that a wrap test be used as a criterion for the ductility of prestressed concrete wire. The wrap test as described, suggests that if the wire can be wrapped on a mandrel equal to 4 times the wire diameter, the material will be satisfactory.

In a wrap test over a 4 diameter mandrel the exterior fibers of the test specimen are elongated 20%. A wire wrapped on its own diameter is stretched 50% at the exterior wire surface. At first it is somewhat difficult to understand how a material which fractures in a tensile test after only 6% elongation can stretch 20% in a wrap test.

An examination of the elongation which takes place in a 10" gage length of a tensile specimen will show that possibly 95% of the gage length will have stretched only 5% to 6% but at the point of fracture the elongation may be over 50%. For example, a wire which has a reduction of area of 50% at the point of fracture must necessarily elongate 100% in this same area. We should realize, therefore, that the nominal elongation value is an average for the entire gage length and does not indicate the large amount of local plastic deformation that takes place at the fracture.

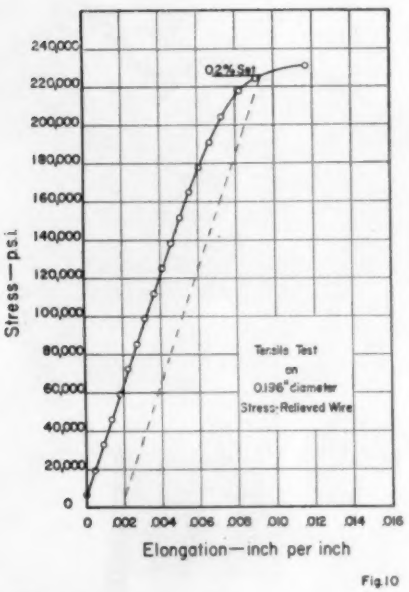
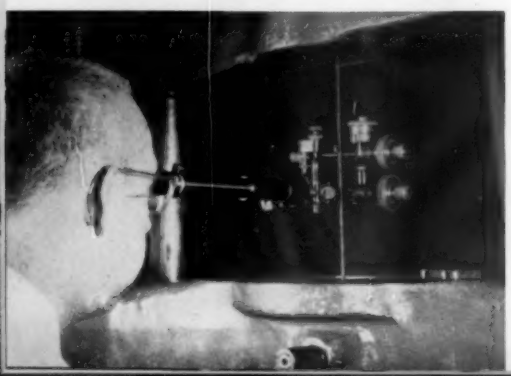


Fig. 9—Stress-strain test on wire.



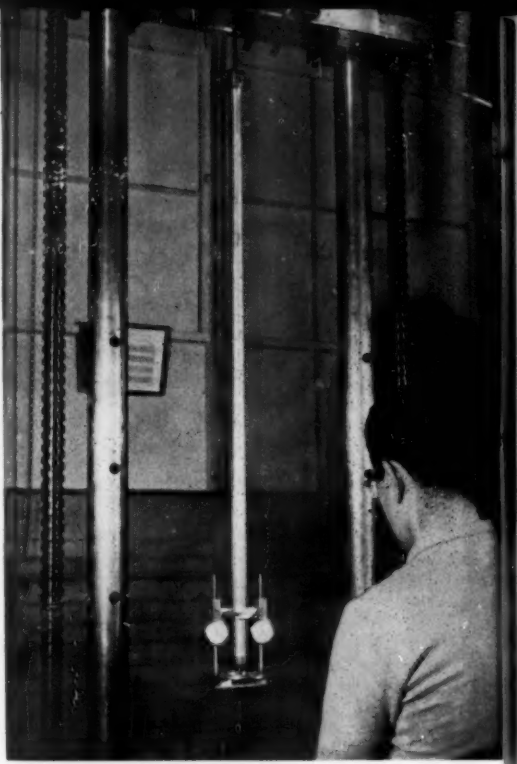


Fig. 11—Strand extensometer.

The wrap test is of value, particularly when surface defects are present. However, we have found that the best index of ductility has been the reduction of area as determined by the tensile test. Experience in the field has indicated that wires having a low reduction of area may not be satisfactory under the high sustained stresses used for pretensioning. Wire may meet all of the other requirements but based on past experience we now reject all material having a reduction of area of less than 30%. This requirement allows for a good margin of safety and our inspection procedure includes the determination of the tensile strength, elongation and reduction of area on both ends of each coil of wire processed.

As we shall illustrate, the wrap test does not always reflect low ductility, particularly when the low ductility is not the result of a surface condition.

A coil of prestressed concrete wire which had been rejected for low reduction of area was tested for its ductility as measured by the wrap test. The reduction of area of this

particular material was found to be only 17% and the tensile fracture showed an abnormal condition. Instead of a normal cup-cone type fracture, the wire exhibited a sharp conical type fracture, as illustrated in Fig. 6.

This same wire could be wrapped on a mandrel approximately 3 times the wire diameter without fracturing. The method of making the wrap test is shown in Fig. 7. A second wrap test was then made on its own diameter and, as shown in Fig. 8, both wrap tests were satisfactory. These tests indicate that the wrap test will not always eliminate defective material.

The wrap test was inadequate in this instance because the surface quality of the wire was satisfactory but the very center of the wire was below standard. The center of

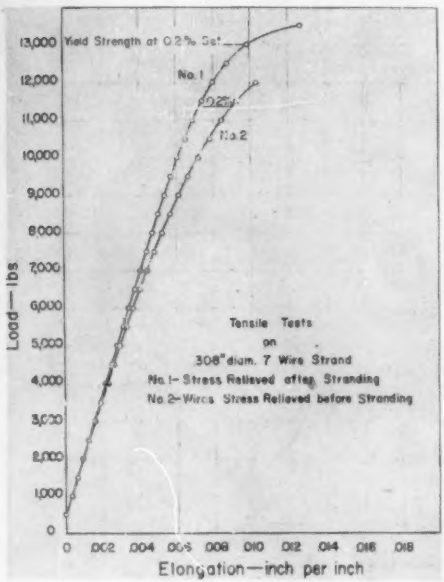


Fig. 14

the wire lies on the neutral axis of the specimen during the wrap test and therefore is not stressed in tension.

The reduction of area test is affected by both interior and surface defects, and for this reason is recommended as a ductility requirement for prestressed concrete materials.

3. Yield Strength

Having discussed the strength and ductility of wire and strand, we should now consider their behavior under working loads. In conventional reinforced concrete design, it is general practice to keep the design stresses within the elastic range of the material and considerably below the stresses at which excessive yielding takes place in the steel.

In contrast to reinforcing bars, steel wire and strand do not exhibit a definite yield

Fig. 12—Top clamp on strand extensometer.

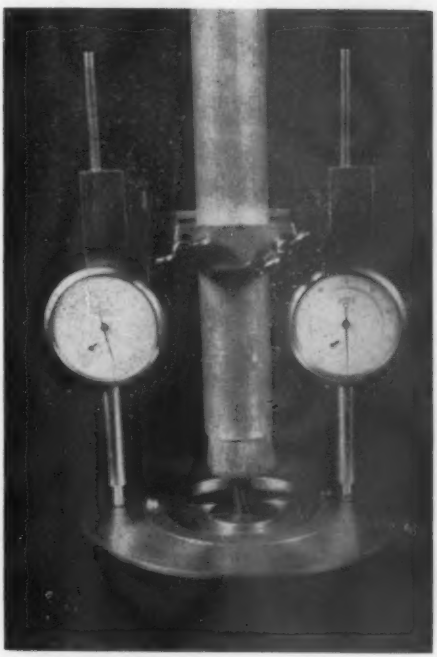
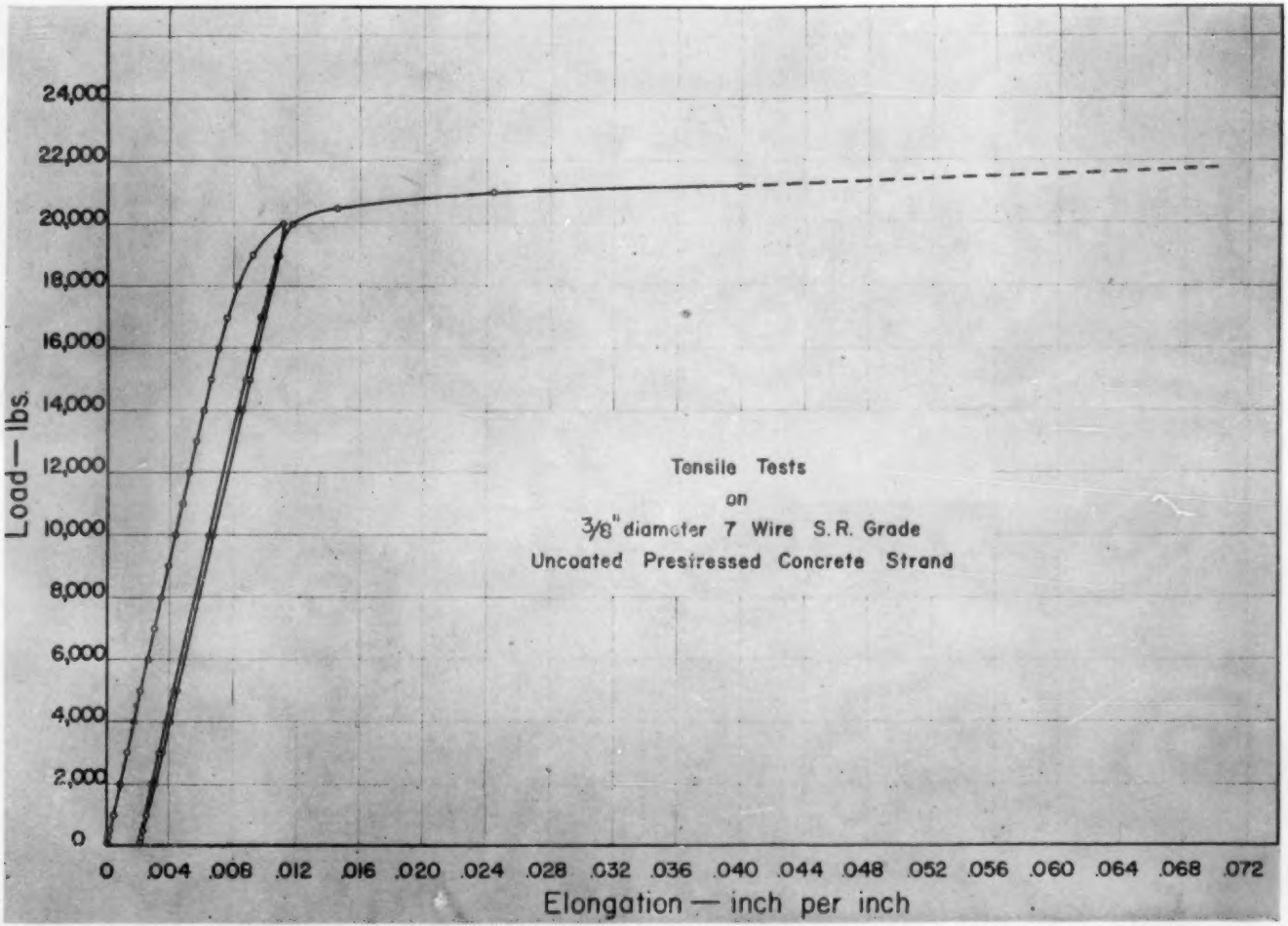


Fig. 13—Dial gages and base plate of strand extensometer.


Load — lbs.
24,000
22,000
20,000
18,000
16,000
14,000
12,000
10,000
8,000
6,000
4,000
2,000
0
0 .004 .008 .012 .016 .020 .024 .028 .032 .036 .040 .044 .048 .052 .056 .060 .064 .068 .072
Elongation — inch per inch
Tensile Tests
on
3/8" diameter 7 Wire S.R. Grade
Uncoated Prestressed Concrete Strand

point but yield in such a gradual manner that their plastic behavior must be determined from a stress-strain curve. The method of measuring the stress-strain relationship of wire is illustrated in Fig. 9, where a 2" gage length optical extensometer is being used to measure the elongation of the wire. After obtaining the required data, the stress-strain curves are then plotted on a graph, as illustrated in Fig. 10. The yield strength in this instance was determined by the offset method using an arbitrary value of 0.2% or .002 inch per inch permanent strain.

The offset method assumes that if the load is released from any point on the curve, the stress-strain relationship will be described by a line drawn parallel to the straightline portion of the original stress-strain curve.

The yield strength is thus determined by constructing a line from zero stress at a strain of .002 inch per inch parallel to the straightline section of the original curve. The yield strength is the stress at which the constructed line intersects the original curve.

Other arbitrary values, such as 0.1%, can be used for determining the yield strength but the 0.2% value is more frequently used. The yield strength should represent that stress at which the strain increases very rapidly when the yield strength is exceeded. A prestressed concrete structure when loaded beyond this value would develop large permanent deflections.

The tentative specifications require that the minimum yield strength of wire be 80% of the specified ultimate tensile strength. This requirement, together with the specifications for strength, elongation and reduction of area, completely define the physical characteristics of stress-relieved prestressed concrete wire, and no additional tests are necessary.

The yield strength of 7-wire strand is also determined in a similar manner except that a different type extensometer is used. A gen-

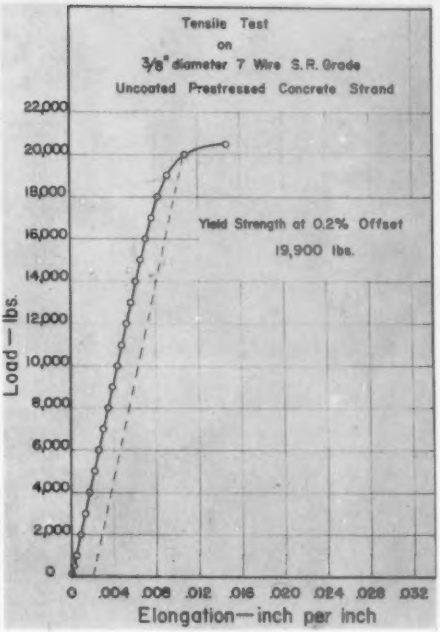
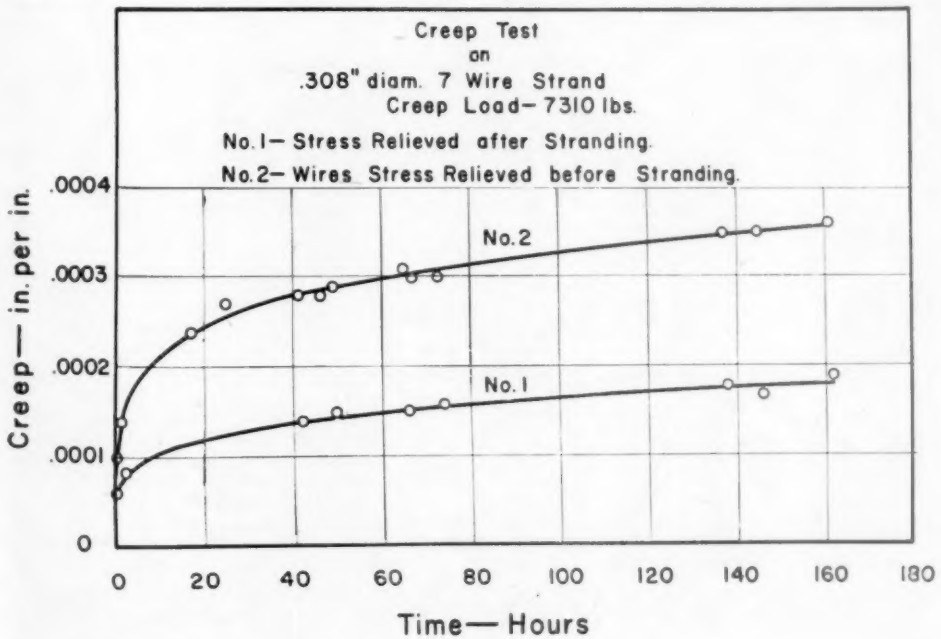


Fig. 16

Fig. 17



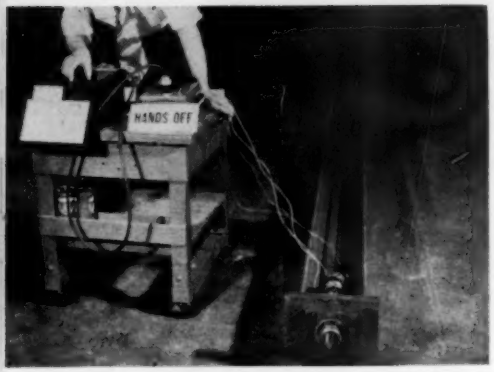


Fig. 18—Relaxation test on strand.

eral view of the strand extensometer is shown in Fig. 11. The instrument consists of a metal tube which is attached to the strand at the upper end by means of split collars inside a flexible coupling, as shown in Fig. 12.

The lower end of the extensometer consists of a flat plate which is attached to the strand at a point 50" below the top clamp (Fig. 13). The elongation in the 50" gage length is measured by two 2" travel dials reading to .001", which is equivalent to a strain of .00002 inch per inch.

A typical load-elongation curve obtained in this manner is illustrated in Fig. 14. The yield strength was determined in the same manner as used for the single wire test.

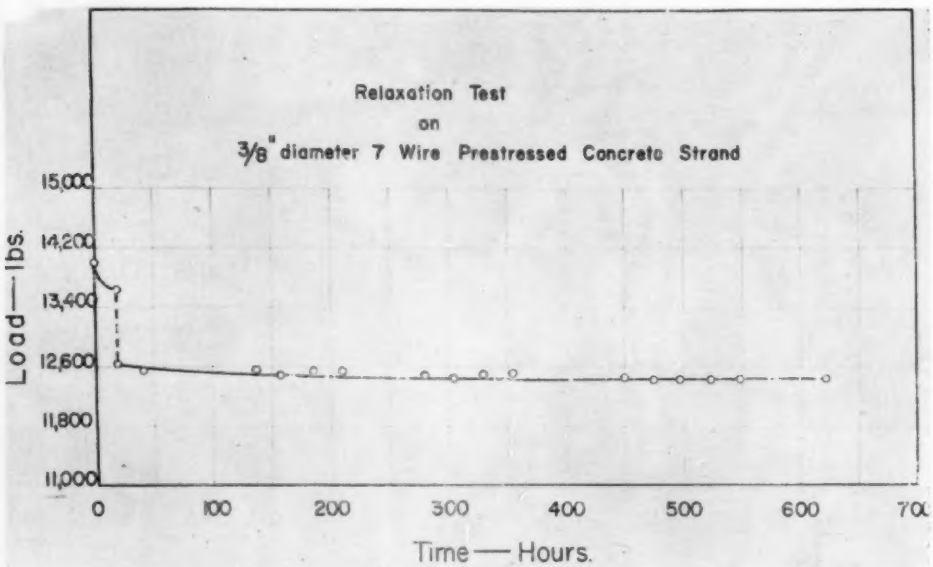
Because the highest plotted value on the load-elongation curve is above the required tensile strength, it is often interpreted as representing the ultimate load. In order to answer this and other questions we are presenting Fig. 15, the data for which was obtained in the following manner:

An initial load of 100 pounds was applied so that the strand would be rigid enough to hold the extensometer steady. Elongation readings were then observed up to a load of 14,000 pounds after which the load was reduced to 100 pounds. During this loading cycle the strand behaved elastically as the elongation readings were the same on both loading and unloading.

The strand was then loaded to 20,000 pounds, which was approximately the yield strength of this particular specimen. At this load the steel was deforming plastically and after reducing the load again to 100 pounds we found that the strand did not return to its original length.

Upon reloading the strand to 20,000 pounds the elongation values traced a different line and formed what is known as a hysteresis loop. In loading beyond 20,000 pounds the strand continued to stretch at a very rapid rate, and after 4% elongation the range of the extensometer dials was exceeded. The total elongation at fracture as indicated by the curve was from 6 to 8%. The large amount of stretch between the yield strength and ultimate strength makes the stress-relieved strand an ideal material for prestressed concrete.

The high elastic properties of strand are obtained by stress-relieving the finished strand and not by making the strand with stress-relieved wire. If the latter method is used, the elastic properties of the stress-



relieved wire are destroyed during the stranding operation, as illustrated in Fig. 16 where the load-elongation curves for the two types of strand are presented. Up to loads included in Fig. 16 the curve representing strand made with stress-relieved wire is very similar to strand not stress-relieved during any manufacturing stage.

The creep properties of the material stress-relieved after stranding were also considerably better than the strand manufactured with stress-relieved wire, as shown in Fig. 17.

4. Creep and Relaxation

The subject of creep and relaxation of prestressed concrete materials has been discussed to considerable extent in previous meetings on prestressed concrete. At this time we would like to present the results of a relaxation test, which, to some extent duplicates the loading conditions that might be placed on a ⅜" diameter strand in a pretensioned beam.

The loading cycles were as follows:

a. Pretensioned to 14,000 lbs.
b. Ends fixed for 18-hour period (during which time relaxation took place)
c. Load intentionally reduced 1040 lbs., or 13,000 p.s.i.
(The 13,000 p.s.i. was obtained from the Bureau of Public Roads Criteria, which provides a loss of 3,000 p.s.i. for shrinkage in the concrete that takes place during the first 18 hours plus a steel stress equal to 5 times the concrete stress, which was assumed to be 2,000 p.s.i.)
d. Ends fixed and relaxation of the strand measured.

The above relaxation test can be readily made in a mechanical type testing machine or in a test setup as illustrated in Fig. 18. The ⅜" diameter strand specimen was approximately 20 feet long and was held at each end with reliable grips. A steel "H" beam was used to take the thrust developed by the strand which was tensioned by a nut and screw arrangement on the end of a calibrated dynamometer.

The result of the relaxation test is shown in Fig. 19. The test indicated that under the initial prestressing load of 14,000 lbs. the loss in stress over the first 18 hours was only 360 lbs. After the load was intentionally reduced 1040 lbs. to simulate the transfer of stress to the concrete, very little stress loss was observed. In practice, therefore, we may assume that any appreciable stress loss in the strand after transfer of load would be due to shrinkage or plastic flow of the concrete.

5. Effect of Welds

During the fabrication of strand it is necessary to make welds in the individual wires. These welds are necessary in order to make a long continuous length of strand. In any one wire position in a 7-wire ⅜" diameter strand these welds occur at approximately each 10,000 feet. With 7 wires in the strand, however, it is possible to have several welds in the same section of strand if suitable care is not taken during the stranding operation. In practice, the welds are limited to one in any 150-foot length of strand.

Special lengths of strand were fabricated containing from one to four welds in each test length. Only one weld was made in each wire and where more than one weld was in the strand the welds were approximately one foot apart. The results of the tensile tests are presented in Table II.

The Effect of Welds on the Tensile Strength of ⅜" Dia. 7-Wire Strand

Test No.	Number of Welds	Tensile Strength Pounds	Per Cent of Catalog Strength
1	0	22,300	111.5
2	1	19,400	97.0
3	1	18,800	94.0
4	2	16,400	82.0
5	2	17,300	86.5
6	3	17,000	85.0
7	3	16,800	84.0
8	4	16,000	80.0
9	4	15,600	78.0

TABLE II

As indicated in Table II, the tensile strength of the strand generally decreased as the number of welded wires increased. Since welds are limited to one in any 150 foot length of strand it should be possible to safely apply the recommended pre-tensioning load of 14,000 pounds.

Welding is not recommended for use in the manufacture of larger diameter single wires for prestressed concrete. Any high temperature heat treatment such as welding will seriously affect the properties of high carbon steel wire. The users of these high strength steels should be cognizant of this heat effect and make sure that all necessary precautions are taken in the field.

ECONOMICS OF PRESTRESSED CONCRETE...

Fig. 1—Lake Ponchartrain Bridge under construction, a 24-mile pretensioned concrete bridge, 56-ft. spans, Louisiana. (Palmer and Baker, Inc., Mobile, Alabama)

By T. Y. Lin

Professor of Civil Engineering

University of California, Berkeley

INTRODUCTION

When prestressed concrete bridges were first being built in this country around 1950, there were over-enthusiastic engineers who believed that this new method would soon monopolize the field of bridge construction. On the other hand, there were also pessimists who doubted the economy of any complicated procedures and advanced techniques. Things have cleared up during the past few years, however. Individual instances as well as general trends have indicated the definite economy of such construction under certain conditions, and the doubtful economy under others. It is now safe to say that, for bridge construction, prestressed concrete is taking its proper place among the family of major construction materials, which formerly consisted only of steel, timber, and reinforced concrete. Being a new type, it will naturally replace the conventional materials more and more as engineers and contractors learn to master the techniques required for its application.

INDIVIDUAL INSTANCES

One way to study the economics of prestressed concrete bridges is to take individual cases where actual bids were taken on alternate designs for the same crossing. While such comparisons are often enlightening, they can also be misleading. For example, a contractor might increase his bid to cover the risk of trying out a new type of construction; or he might have been over-optimistic in his bid and actually taken a loss in the job. There have been times, too, when keen competition among the various prestressing systems forced the price down to an unusually low level, and the low bid was no indication of the basic economy of prestressed concrete.

There have also been instances where bids on the alternate prestressed concrete design were higher than the conventional design of steel or reinforced concrete. Neither was this necessarily a proof against the economy of prestressed concrete. Engineers not acquainted with its theory and practice often tend to be over-conservative in the design.

*This paper is also published in *Proceedings, California Street and Highway Conference, 1956.*

Contractors, not familiar with the techniques of prestressing, also tend to add extra contingency costs to the estimate and may not know how to economize in the construction of such bridges.

At times, the owner or engineer has chosen prestressed concrete bridges over other types not because of reasons of economy. The owner might have been willing to pay a slight premium for trying out a new type of construction. The bridge might have been at a special location where shallow depth, light weight, or other requirements dictated the use of prestressed concrete. Hence its adoption may not be a definite proof of its general economy.

BASIC COST DATA

Rather than taking isolated instances as a basis for comparison, it is possible to study the general economic possibilities by examining some basic cost data. A simple approach is to compare prestressed with reinforced concrete. For reinforced concrete bridges, we are using cylinder strength of about 3,000 psi. For prestressed concrete, we use 5,000 psi. Hence the strength is 1.67 times higher. But the cost-in-place of such stronger concrete is no more than 15% higher. Hence, the cost per unit of load carried by the bridge is lower for prestressed concrete. There are, of course, other factors which affect this cost ratio. The smaller dead load of the prestressed bridge and the effectiveness of the uncracked section of concrete tend to increase the relative economy of prestressing while the practical requirement for minimum dimensions often narrow the gap between reinforced and prestressed construction.

Steel bars for reinforced concrete are designed with a stress of about 20,000 psi. Wires and strands for prestressed concrete have a working stress of around 130,000 psi. The cost-in-place of reinforcing steel is $0.11 per lb., of post-tensioning steel about $0.60 per lb., and of pre-tensioning steel about $0.33 per lb. Since the strength of prestressing steel is 6.5 times that of reinforcing steel, while the unit cost is only 3 to 5.5 times higher, the economy of prestressing steel is self-evident. The cost for each case, however, will vary greatly depending upon the quantity required, the type of steel and anchorage, the location and grouting of the tendons, the additional requirement for non-prestressed steel, and other factors. Hence the values mentioned above should be taken only as an indication of the general trend, not necessarily applicable to all cases.

Formwork is the one major item that works against the economy of prestressed concrete. In order to save concrete and steel, it is often necessary to use relatively complicated forms. Hence, for most bridges the cost of formwork per unit volume of concrete is higher for prestressed than for reinforced work. If a special form is to be built for a prestressed beam and used only once or twice, the cost of forms may be so excessive as to more than offset the savings in concrete and steel. On the other hand, if the form is used many times over, its cost is minimized and savings result.

Falsework for bridges is generally cheaper for prestressed concrete, because it is lighter. Furthermore, the lightness of the elements often makes possible the use of precasting, thus cutting down the amount of falsework.

COMPARISON OF REINFORCED CONCRETE AND STEEL BRIDGES

Comparison of Reinforced Concrete and Steel Bridges

From the analysis of above cost data, it becomes apparent that the economy of prestressed concrete lies in the savings in cost of concrete and steel at the expense of in-

Fig. 2—Los Penasquitos Bridge, 6 spans at 28-ft. City of San Diego, California. (Southwest Structural Concrete Corporation, San Diego)

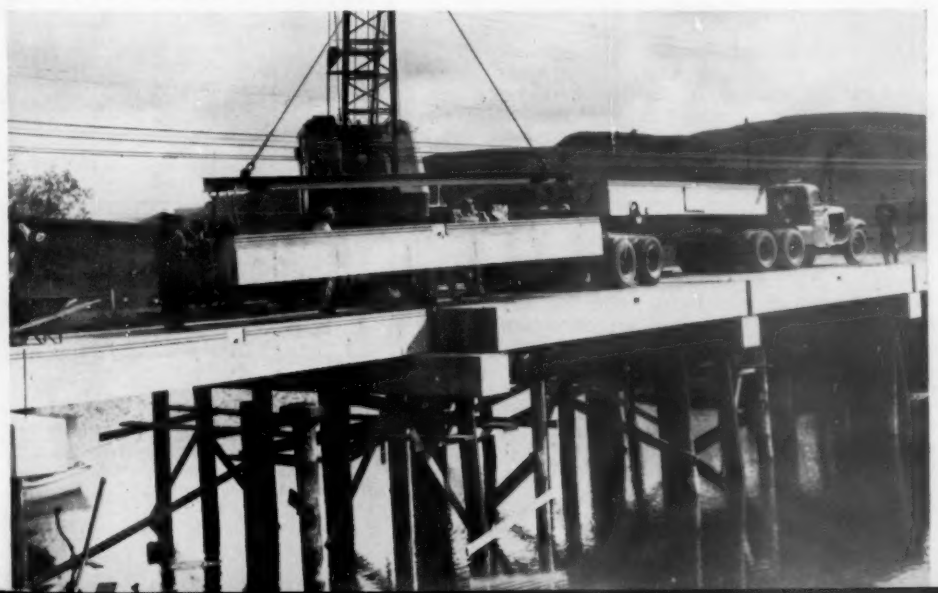


Fig. 3—Eureka Bridge of prestressed concrete, California Division of Highways. (Ben C. Gerwick, Inc., San Francisco)

creased formwork. In order to get an appreciable resultant saving, the multiplicity of units becomes a principal consideration. Take, for example, the Lake Pontchartrain Bridge, a 24-mile bridge now under construction in Louisiana (Fig. 1). For the purpose of bidding, two designs were prepared, one of prestressed concrete and the other of steel. Bidders were informed that award would not be made on the steel design unless it was at least $1,500,000 cheaper than the prestressed concrete design, with this differential representing the capitalized value of estimated painting costs for the steel bridge. When the bids were opened in June, 1954, the lowest bid on the prestressed concrete design was $27,600,000, while that on the steel was $33,517,507. This saving of nearly $6 million was possible because this was a bridge 24 miles long with thousands of identical spans. Each deck span was precast as a unit, 56 ft. long and 33 ft. wide. High economy was obtained by re-using forms and centralizing production; $4.5 million were spent on establishing a plant to produce these prestressed piles and deck units.

For smaller bridges, economy is assured only when some pre-tensioning yard is nearby, or where transportation of precast units is inexpensive. The Los Penasquitos Bridge in San Diego, California, is typical of a small bridge, having only 6 spans of 28 ft. each Fig. 2). This prestressed concrete bridge was bid at the same price as the steel design. Economy was effected in the time required for construction. The entire superstructure was erected in only 14 hours of crane time. Since there were not enough repetitions, simplication of formwork was imperative. The stems for the channel sections were separately cast flat on the pre-tensioning bed and then joined together by casting the top flanges. Such separate castings may not be necessary if bridge sections are standardized so that one set of forms can be used for many bridges.

Where convenient water transportation is available, units can be made in a plant even at some distance away and shipped to the bridge site economically. Fig. 3 shows the Eureka Bridge, designed by the California Division of Highways which was made in Ben C. Gerwick's plant at Petaluma, 200 miles from the bridge site. Barging along the Pacific Coast made it possible to ship the units economically.

While methods of design and construction, means of transportation, and procedure

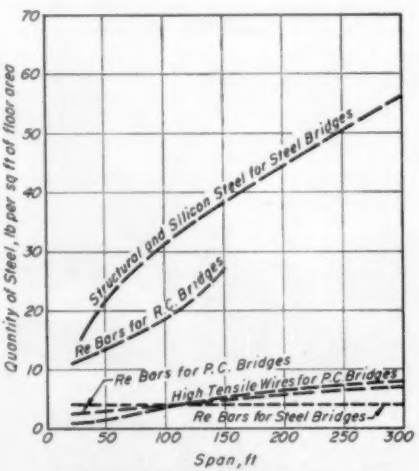


Fig. 4 AVERAGE QUANTITY OF STEEL FOR BRIDGE SUPERSTRUCTURE
(H20-S16 loading)

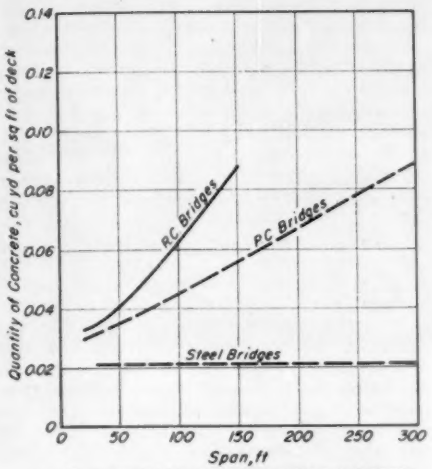


Fig. 5 AVERAGE QUANTITY OF CONCRETE FOR BRIDGE SUPERSTRUCTURE (H20-S16 loading)

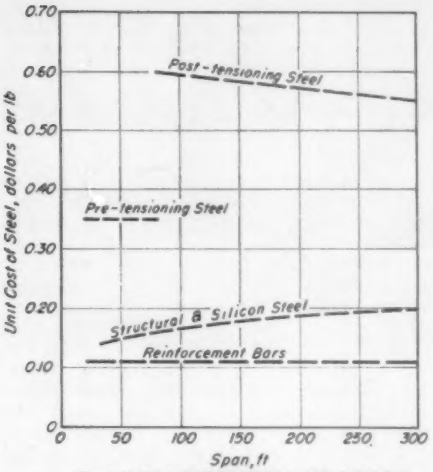


Fig. 6 UNIT COST OF STEEL FOR BRIDGE SUPERSTRUCTURE (Cost in place including accessories)

of erection vary with each bridge, it is possible to get approximate values representing the average conditions in order to convey some idea concerning the economics of prestressed concrete as compared to reinforced concrete and steel bridges. No attempt will be made to compare a prestressed concrete bridge with a timber one. By the initial cost alone, a timber bridge, when feasible, will often be the cheapest; but it may not be the most economical when the cost of replacement is taken into consideration. Again, the cost for painting steel bridges will often tip the balance in favor of prestressed concrete.

Considering the initial cost alone, it is possible to compare the economy of various types by their quantity and price of steel and concrete. Fig. 4 shows the average quantity of steel required for bridge superstructure of different span lengths, designed for H2O-S16 loading. Fig. 5 shows the corresponding average quantity of concrete. For a given bridge, the quantities required may often vary from those in the figures by 10% to 20%, depending upon the layout and cross-section of the bridge, the strength of steel and concrete, and other factors.

Based on the in-place price of steel and concrete (Figs. 6 and 7) and on the quantity charts (Figs. 4 and 5), the cost of bridge superstructure in dollars per sq. ft. of floor is shown in Fig. 8. It must be

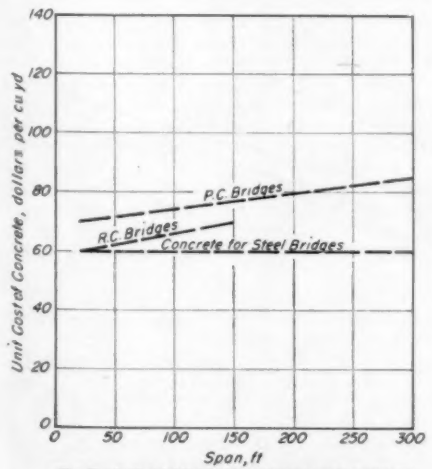


Fig. 7 UNIT COST OF CONCRETE FOR BRIDGE SUPERSTRUCTURE (Cost in place including form work and false work)

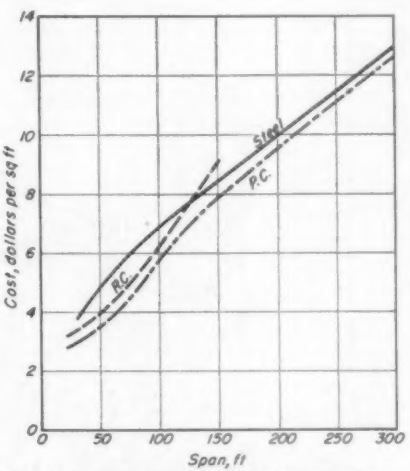


Fig. 8 COST OF BRIDGE SUPERSTRUCTURE (H20-S16 loading)

pointed out that unit price varies from place to place, from time to time, and from bridge to bridge. Hence each bridge must be considered by itself. The values given are meant to represent the so-called average conditions in this country, and must necessarily be modified for individual cases.

A look at Fig. 8 will reveal that the average cost of prestressed bridges is somewhat below – and at least comparable to – steel and reinforced concrete bridges of all span lengths up to 300 ft. and beyond. These curves, being representative only of the average conditions, should not be taken to mean that prestressed bridges are cheaper under all conditions. Nor should they be taken to mean that the cost differential is always as limited as indicated by the curves. Variations in local conditions are often sufficient to tip the balance in favor of one or the other type, so that sometimes one and sometimes the other will be the most economical – perhaps by a wide margin.

In the above curves, the use of lightweight deck and alloy steel has been considered in the quantity curves for the steel bridges, but only ordinary sand and gravel concrete has been used for the prestressed and reinforced bridges. The use of lightweight for longer spans will place concrete bridges in a more favorable position than indicated herein.

PRE-TENSIONING vs. POST-TENSIONING

In prestressed concrete, pre-tensioning is a procedure wherein the tendons (prestressing steel) are tensioned against heavy abutments before the concrete is placed. Post-tensioning designates a procedure wherein the tendons are tensioned against the concrete after it has hardened. The economy of the pre-tensioning type lies in the general absence of end anchorages, the elimination of sheathing and grouting or greasing, and the use of long-line production – thereby saving in tensioning and concreting.

When a job is large enough it may justify special pre-tensioning beds and devices. For small bridges, pre-tensioning is economical when a plant is nearby or where the units can conveniently be transported from the plant to the site. Bridge girders requiring higher prestressing force than the capacity of the pre-tensioning bed may be built economically by combined pre-tensioning and post-tensioning. Long-span bridges may be too heavy to be precast in the bed and may have to be post-tensioned in place. Continuous spans are often cast in place, thereby requiring post-tensioning. Partially continuous spans, where the units are first pre-tensioned or post-tensioned and then made continuous by post-tensioning or reinforcing bars, are often found to be economical where the saving in falsework is desirable or necessary.

One disadvantage in pre-tensioning lies in the fact that many pretensioning plants are equipped to tension only straight wires or strands. Some plants, however, have beds made for the purpose of harping the tendons. The economy of pre-tensioning units will prevail in the longer spans when such harping is possible. Instead of harping, the ten-

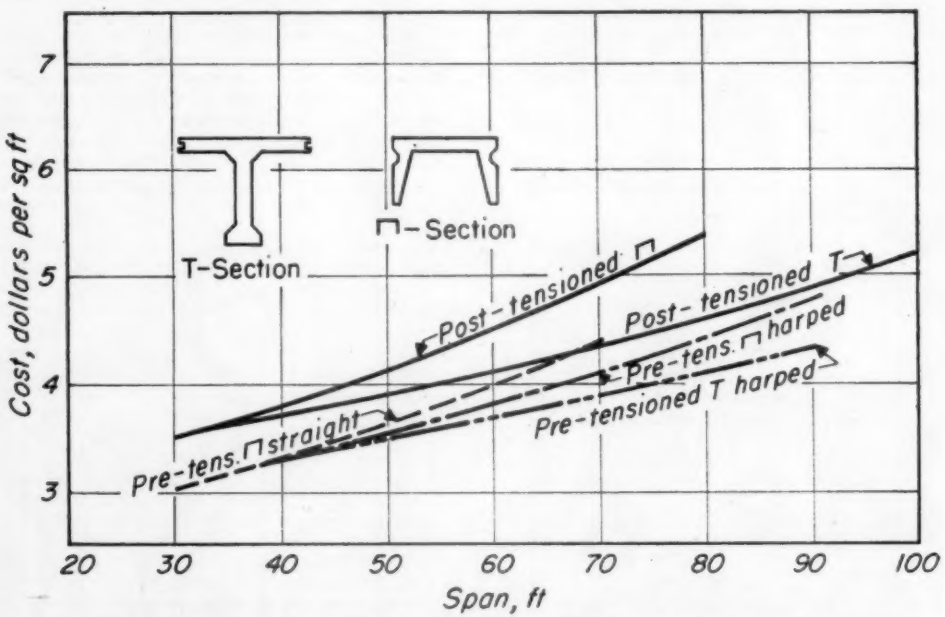


Fig. 9 PRETENSIONING vs POST-TENSIONING
(H20-S16 loading)

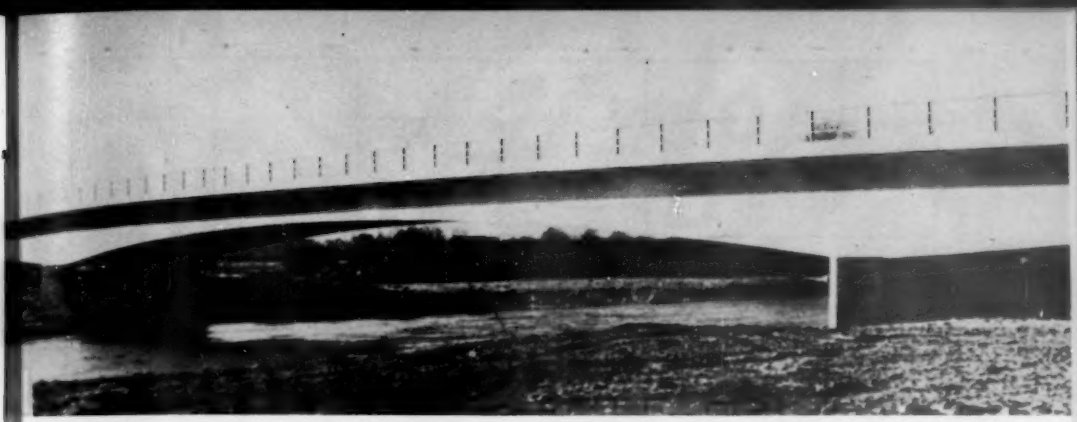


Fig. 11—Prestressed-concrete continuous bridge over Leine, Germany, 107-230-107 ft. spans. (Phillip-Holzmann, Frankfurt)

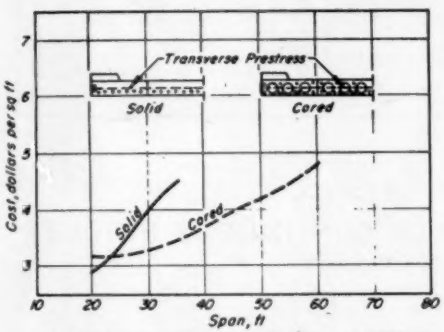


Fig. 10 COST STUDY: PRE-TENSIONED SLAB BRIDGES. (H20-S16 loading)

dons can be greased and wrapped to destroy the bonds at the ends, or part of the tendons pre-tensioned and the remainder post-tensioned along a curved profile.

Relative cost of post-tensioning against pre-tensioning is shown in Fig. 9. Since actual cost varies so much in different places and at different times, these values should be taken only as an approximate guide. It may be observed from Fig. 9 that the post-tensioned channel section appears to be more costly except for very short spans. This is not always true when the relative simplicity and cost of joining the sections is taken into consideration. When the depth of a bridge is not limited, the T section with straight stems (without bottom bulb) is often the most economical.

For the cost of pre-tensioned units, the T sections are generally cheaper than the channel. Harped tendons always yield a more economical design than straight ones. A comparison of pre-tensioning against post-tensioning reveals that post-tensioned T sections can be cheaper than pre-tensioned units using straight tendons. But if harped tendons are adopted in pre-tensioning, the only limit of economy may be the size and length of the units as controlled by transportation and erection requirements.

The cost of bridges using composite construction will not be discussed here. Early prestressed bridges built in this country were almost exclusively of this type, with the stems precast and post-tensioned and the floor slab cast in place to form the top flange and to integrate the units. More recently, the stems were pre-tensioned. Where heavier units can be handled, composite action has given way to T sections and precast units such as for the Lake Pontchartrain Bridge.

PRE-TENSIONED SLAB BRIDGES

Some simple forms of pre-tensioned slabs have been developed for bridges of short spans. A solid section is seldom economical except for very short spans. Though the cost of formwork is reduced to a minimum, much volume of concrete and steel is required. In fact, it is doubtful whether solid sections will be able to compete with ordinary precast reinforced concrete slabs.

Bridge sections using hollow paper cores have been widely used. The cost of the cores just about balances the cost of the concrete replaced by them; but they also save

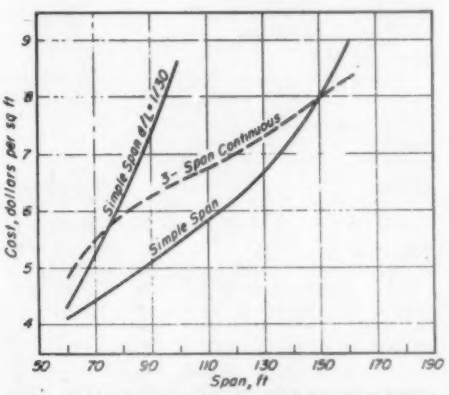


Fig. 12 COST STUDY: SIMPLE vs CONTINUOUS SPANS (H20-S16 loading)

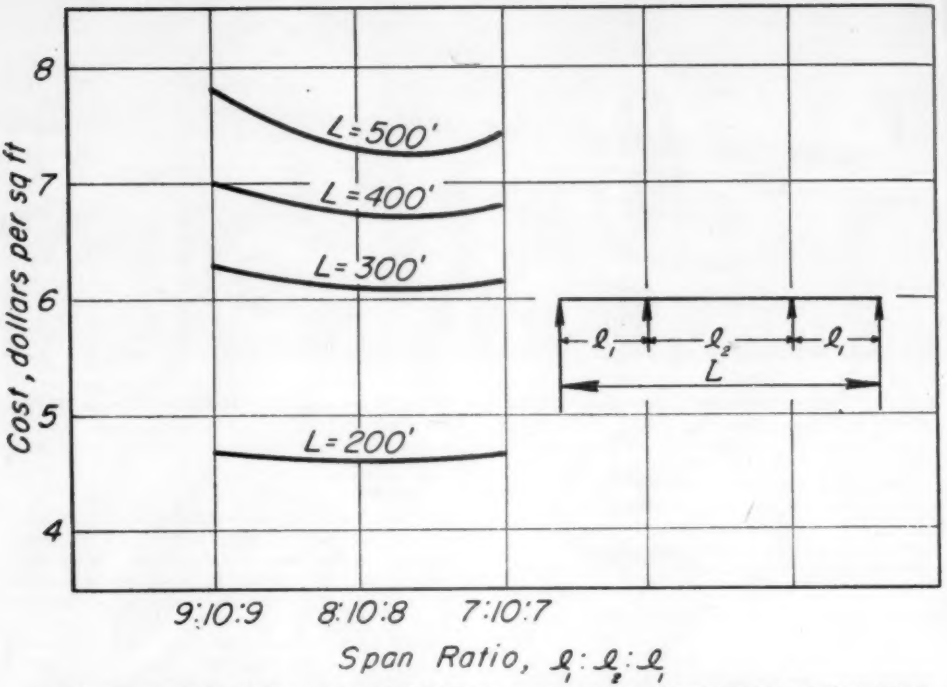


Fig.13 ECONOMIC SPAN RATIO FOR CONTINUOUS BRIDGES (H20-S16 loading)

steel and reduce the weight of the units. Slab bridges have relatively shallow depth, but where depth is not limited they cannot compete with the T sections for spans over 30 or 40 ft. Relative costs of solid and cored slabs are shown in Fig. 10.

CONTINUOUS BRIDGES

For multiple-span prestressed bridges, it is known that continuity may help to reduce the cost. When the bridge is precast as simple spans but later joined together over the supports to achieve continuity, the bridge is termed as partially continuous. Where falsework is expensive, such construction may be found to be economical for medium span lengths.

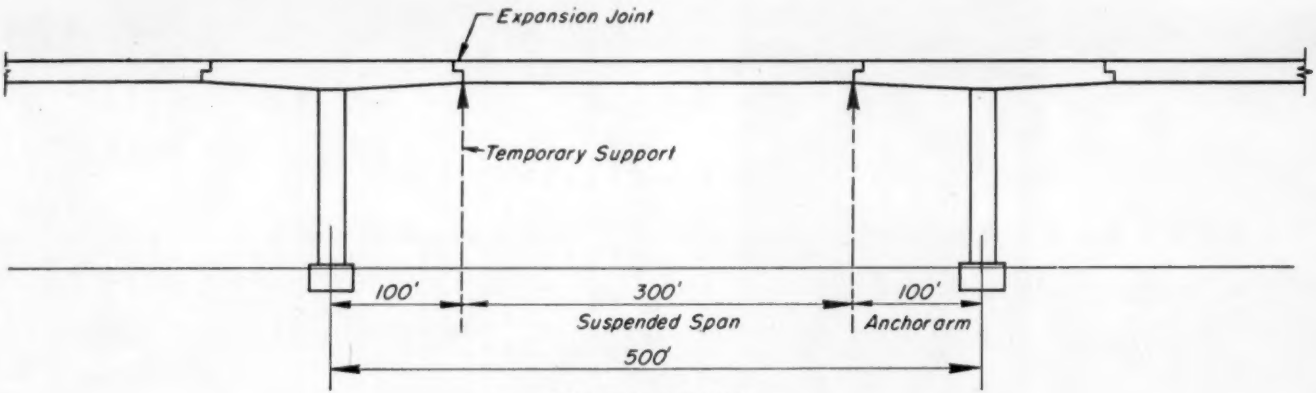
Fully continuous bridges are generally cast in place. Continuity saves both steel and concrete, and gives a shallower depth at midspan; a depth-span ratio of 1/45 for highway bridges is not uncommon. A German bridge over the Leine near Schwarmstedt has spans of 107, 230, and 107 ft. with a depth of 5 ft. at center and 10 ft. 8 in. over the supports (Fig. 11). This type of construction has not yet found popularity in the United States, partly because of the hesitancy of engineers to go into continuous designs.

Fig. 12 indicates the relative cost of continuous and simple spans. Of prime consideration is the question of whether the depth for the simple spans is limited. If the depth-span ratio is limited to 1/30, a continuous layout may be economical for spans of 75 ft. and up. On the other hand, if the simple span can be built of any depth, its economy can be extended up to about 150 ft. No limitation was set for the depth of the continuous layouts in this figure, since the economical depth is shallow enough for most cases.

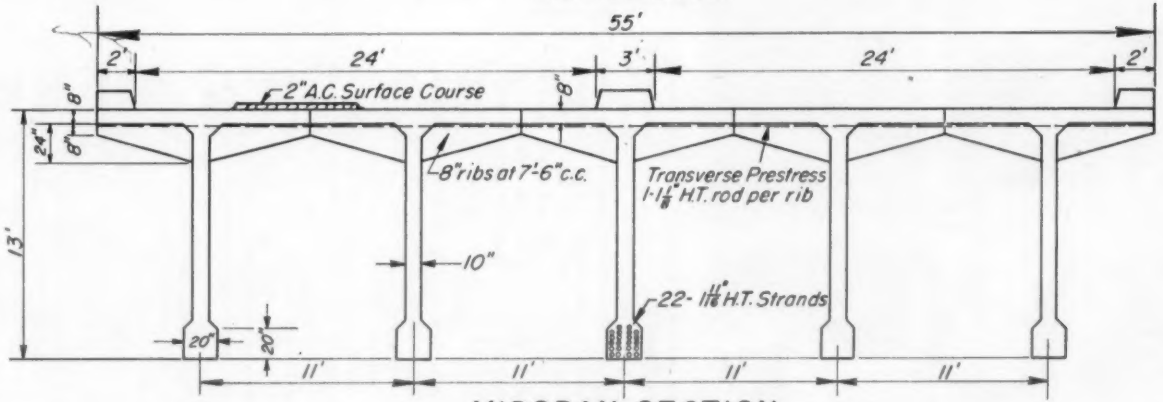
There are many problems involved in a continuous bridge layout. Take, for example, a crossing of length L to be spanned by three spans. Assuming the cost of piers to be the same regardless of their location, what should be the ratio of the span lengths in order to get a most economical layout? This problem is studied and plotted in Fig. 13. Curves are plotted for total length of crossing from 200 to 500 ft. It is noted that the most economical ratio for side to center span lengths is just about 3:4:3 for all cases, although slight variation from that optimum ratio does not entail appreciable additional cost, especially for the shorter lengths of crossing.

LONG SPANS

For very long spans, the most economical layout must be considered together with the method of construction and erection. If a crossing consists of one or only a few spans, in-place construction may be the most economical. For multiple-span layouts, say more than five spans, it is often desirable to



ELEVATION



MIDSPAN SECTION

Fig. 14 LAYOUT FOR PRESTRESSED CONCRETE BRIDGE WITH 500FT SPANS.

(H20-S16 loading)

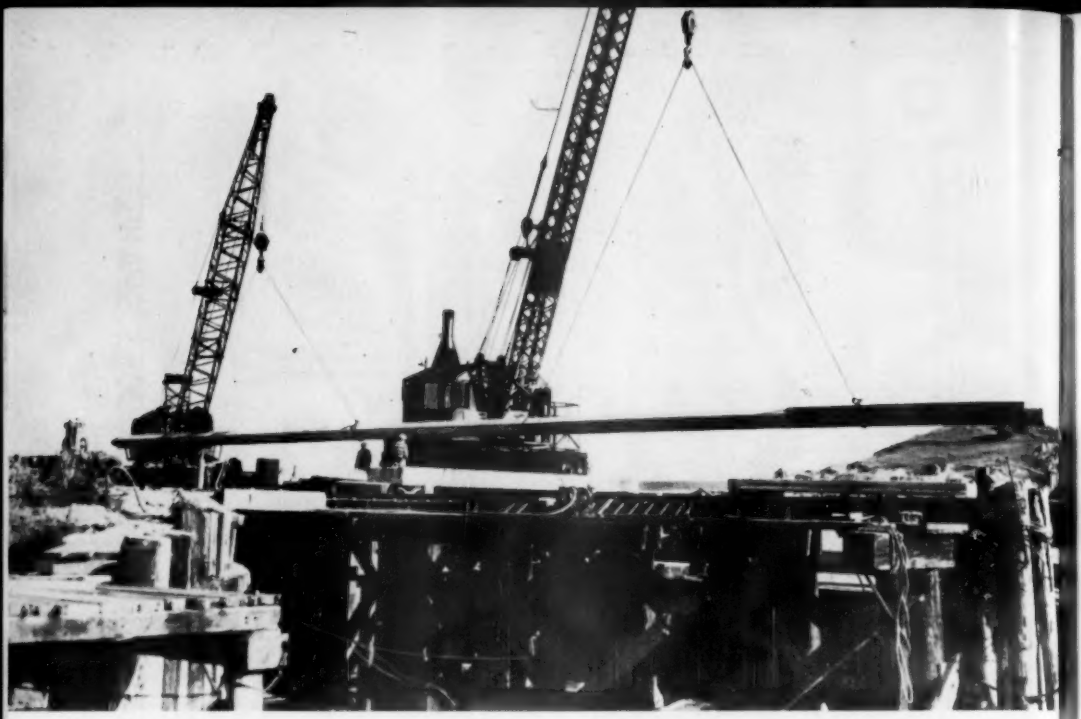


Fig. 15—Pretensioned piles 131 ft. long, California State Harbor Commission. (Ben C. Gerwick, Inc., San Francisco)

consider the precast units, provided the units can be erected economically. Fig. 14 shows a continuous-cantilever layout for a bridge with multiple spans of 500 ft. each, where both the pier and the superstructure units are to be cast on shore and floated into position. Lightweight concrete is used in this design.

This design is predicated on the suppositions that the river is navigable and the units can be barged to the site, and that the number of spans are such as to justify the installation of equipment for the handling of these units, up to 300 tons apiece. Depending upon the circumstances, at least 5 or 6 identical spans are necessary to make this layout economical. The cost of this bridge superstructure is estimated at $19 per sq. ft. of floor area for a bridge with 10 spans, increasing to $25 per sq. ft. for 5 spans, and decreasing to $16 for 20 spans.

FOUNDATIONS

Prestressed concrete piles have been found to be economical for certain bridge substructures. The Eureka Bridge, California, (Fig. 3) used 20-in-square pre-tensioned piles, which were chosen by the contractor against a reinforced concrete pile alternate. Advantages of prestressed piles are the capacity to carry lateral loads, the higher buckling strength, and better resistance against corrosion. They are also easier to handle, thus facilitating driving and hauling. Fig. 15 shows a 20-in. octagonal pre-tensioned pile 131 ft. long for Piers 15 and 17 on the San Francisco waterfront.

CONCLUSIONS

We have indeed just begun to learn the uses of prestressed concrete in bridge construction. For shorter spans, pre-tensioning seems to be the answer whenever plants are nearby or whenever the size of the job warrants the establishment of a special plant. For medium spans, continuous construction, whether partially or fully continuous, may be considered; and post-tensioning is involved in both cases. Layout must take into consideration the site situation and the method of erection. Multiplicity of units and simplicity in erection will help to make prestressed concrete design competitive with steel, even for long-span bridges.

Generally speaking, prestressed concrete bridges may be found to be economical for spans varying from 25 to 500 ft. or more. For bridge foundations and trestle bents, prestressed piles may often be desirable and economical.

It is necessary to emphasize again the fact that while general trends and ranges of economics are discussed in this paper, they should be taken only as reference values which may or may not be applicable to a given location. Each bridge must be studied by itself; no one material is likely to monopolize the entire field of bridge construction.

Acknowledgement is given to Messrs. H. R. May, J. W. Murray, D. C. Perry, W. R. Spillers, K. C. Thornton, and J. A. Willis who contributed toward the preparation of the curves and data used for the various figures in this paper.

a look at PRESTRESSED CONCRETE thus far in California

By Arthur L. Elliott
Bridge Engineer — Planning
California Division of Highways — Bridge Department

Presented at the 1956 meeting of the Highway Research Board

Within the space of the past six years, prestressed concrete in the United States has grown from a brand new idea into an accepted method of concrete construction. This growth, a result of a new application of existing materials and theories, is in itself quite phenomenal. In Europe, during the past ten years, shortages of materials and enforced economies in construction have given prestressed concrete a substantial start. The development in the United States, however, has been somewhat slower to get under way. Its acceptance has been characterized by some hesitancy, both on the part of designers and contractors, mainly because of lack of experience, and reluctance to abandon methods of construction in which they were well experienced and equipped. The reception of prestressed concrete has also frequently been clouded by difficulties and misunderstandings concerning the various anchorage systems and types of stressing units which are offered for use.

However, these six years have seen much progress. Prestressed concrete has become more familiar. This has been reflected in prices for the work. Contractors bid the first prestressed work at relatively high figures because they were uncertain as to what they were getting into. These high prices were at first justified. The equipment was makeshift and no one had had any actual job experience. As a result the work was costly. Experience came rapidly, however, and the quality of the work improved. With increased know-how, the work became easier and the prices dropped.

We cannot yet say that we are at the end of the training and learning period. Each

prestressed concrete job still provides new problems and consequently adds to our knowledge of the subject. Neither can we say that prices have dropped to their minimum level. The drop so far has merely been sufficient to bring prestressed concrete within the range where it can be used occasionally on a straight economic basis without having to justify it because of some other advantage peculiar to prestressed concrete.

Prices have not yet reached the point where prestressed concrete design can be substituted with impunity for conventional concrete design. Contractors still have their own equipment and crews trained for conventional construction. They usually bid rather high on prestressed concrete work for which they must import experts or rent special equipment.

FITTINGS, PATENTS AND SALESMEN

The increasing acceptance of prestressed concrete has brought an attendant rush of manufacturers to get into the field of making the prestressing rods, wire and cables and especially the end connections for the tension members. Almost every conceivable method of fastening the wires and rods has been tried. Many are covered by patents. Of the several systems available, each has its own advantage. The strenuous competition has, in some cases acted to the detriment of the prestressed concrete field as a whole.

On public work, it is a desirable policy not to design for any specific system. Consequently, designs must be prepared with sufficient latitude so that any of the leading prestressing methods may be applied to the work. This often results in oversize beams and end details which are cumbersome or undesirable.

The solution is not yet apparent. A general contractor who finds himself the low bidder on a structure containing prestressed elements immediately is besieged by salesmen offering different prestressing systems. Sometimes these systems and devices are new and untried and the unwary contractor may find that he is paying a sub-contractor to work the bugs out of some new device that may or may not be successful. One can readily see that such tactics do not generate confidence in contractors nor encourage reductions in prices.

Unfortunately, the contractor finds he may not combine the best features of several systems to his own benefit and to the benefit of the job, but he must choose one system to follow through, accepting both its advantages and disadvantages. Because the equipment is specialized and the know-how, still not too widely dispersed, the use of the equipment is often tied in with the use of a particular anchorage system.

These factors, while they are a healthy sign of competition and highly desirable from an idealogical standpoint, have nevertheless acted somewhat to the detriment of the prestressed concrete development.

JACKS AND GROUTING EQUIPMENT

Some very clever and efficient jacks and holding devices have been developed to do the stressing work on the tension members. The jacks are usually made and operated by the purveyor of a certain type of fastening device so that the advantages or disadvantages of a given type of jack are tied to the chosen type of anchorage.

Although the grouting of the cable or rod enclosures is generally considered to be of vital importance either from a corrosion protection standpoint or occasionally from the standpoint of achieving full bond, the grouting equipment is often makeshift and impractical. In the early stages of prestressing work, some jobs became hopelessly bogged down because of inadequate grouting equipment.

The subject of grouting and corrosion protection is susceptible to wide discussion and considerable disagreement. The argument is not pertinent to this discussion. However, it is very pertinent to the economics of prestressed concrete to point out that adequate and efficient methods of grouting the prestressing members must be developed and used generally. This is essential to make prestressed work economically competitive. Many contractors, although the actual prestressing work may be handled by some subcontractor, shy away from bidding this type of work, or else bid quite high, because of difficulties they have seen with the grouting operation. So long as fear, ignorance or inexperience are factors influencing a contractor's bid, the prices for prestressed work will be higher than they should be.

POST-TENSION OR PRE-TENSION?

Our first introduction to prestressed concrete was through post-tensioned members. Before we got very far along the road, however, the economic benefits of pre-tensioning became obvious and suppliers began to equip themselves to manufacture pre-tensioned beams.

This opens up an entirely new and wide field and offers a broad horizon to prestressed work. The possibility is rapidly becoming a reality when standard prestressed concrete beams may be secured from a supplier much as steel or timber beams are now ordered. After some experimenting with various sections, bridge members are now being standardized so that suppliers will find it economical to provide semi-permanent forms for repeated use.

The benefits of casting pre-tensioned beams, slabs and piles on beds as long as 500 feet, steam curing them in place, and then cutting them apart and moving them in as little as 24 hours, have already been realized. Large beams, too, have been simplified by pre-tensioning. Where the needed prestressing force exceeds the capacity of the tensioning bed, it has been possible to pre-tension the beam sufficiently so that it can be moved and then add the remainder of the pre-stressing force by post-tensioning.

We have but scratched the surface of possibilities for pre-tensioned concrete manufactured in a plant and sold as a stock commodity. The basic requirement which many

concrete products manufacturers have been slow to realize is that precast pre-tensioned work requires a precision and attention to detail unknown to concrete work before. With care, excellent work can and is being done. Greater facility on the part of the manufacturers and greater acceptance on the part of the designers will most certainly make this phase of prestressing one of the most interesting developments to watch.

PRECAST CONCRETE

Some thirty years or more ago, the concrete industry suddenly became aware of the possibilities of precast concrete work. There was a period when great efforts were made to utilize precast units to the fullest. The basic idea was excellent but when allowed to run wild, the efforts to precast every part of a structure eventually led to its downfall. Structures laid together like building blocks without adequate joint rigidity, eventually collapsed like a house of cards. The use of precast work fell to a minimum.

Now once again we are discovering the benefits of precasting—the benefits of doing the expensive forming and pouring work in a central yard and hauling the finished product out to the job. In this second revival, we must take care that we do not fall into the snares which beset those who came this way before. Precast work must be designed with full knowledge of the necessity of joint rigidity. Rather than make a structure 100% precast, the designers now must realize that there are practical limits to precasting and present day structures must be designed with some of the concrete to be poured in the field to tie the members together.

ECONOMICS

Because the subject of the economy of prestressed concrete depends so heavily on many of the factors discussed thus far in this paper, it is felt that they should be mentioned. The coverage has been more or less general, with a "once over lightly" treatment. Each of these factors is loaded with possibilities for extended discussion. The purpose of this paper, however, after this brief general discussion of the economic influences, is to make a definite comparison of the costs as realized to date on some California projects, compared to work of a conventional design.

Without further ado, therefore, but with full cognizance of the importance and influence of the factors discussed, we will proceed to analyze the costs within our experience thus far.

CALIFORNIA PRESTRESSSED BRIDGES

Because of the newness of prestressed concrete and the inexperience of the contractors in working with it, California's five years of experience do not offer too much conclusive evidence as to the comparative cost of prestressed and conventional concrete construction.

Since September 1950 when bids were taken for the construction of the Arroyo Seco Pedestrian Bridge in Los Angeles — California's first prestressed structure — twenty projects have been built using various types of prestressed units. A similar number of projects are being designed for construction within the next three years.

Prices are available for the twenty completed structures and in the following analysis an attempt has been made to compare each prestressed concrete structure with a similar structure of conventional design. These twenty structures are composed of a variety of types of prestressed elements. The twenty projects were divided as follows:

Number of Structures	Prestressing Application
2 Pedestrian Overcrossings	T-shaped Girders
1 Pedestrian Overcrossing	Slab
6 Structures	T-shaped Girders
1 Structure	Inverted T-section
2 Structures	I-shaped Girders
3 Structures	Precast Channel Sections
1 Undercrossing	Slab
2 Structures	Box Girders
2 Miscellaneous Structures	Caps and Beams

Prestressed concrete is not practically nor economically adapted to all situations. Like all other materials, there are special advantages of the system which make it more usable in one situation than in another. It is of special interest, therefore, to analyze the reasons for selection of prestressed concrete designed for these various structures.

In seven of the cases, prestressed and precast concrete was chosen in order that the beams might be swung into place either over water or over traffic with a minimum of interference. Economy and elimination of falsework were the main benefits dictating this choice.

In six cases the prestressed design was chosen for separation structures in order to reduce the depth of the member to a minimum and maintain the maximum clearance between roadway grades. Precast and prestressed construction was chosen in three cases because of the advantages and saving possible by the mass production of a large number of similar units. An attendant saving is to be found in construction without falsework.

Three more bridges were made precast and prestressed because of their remote location. It was possible to precast the elements of the bridge at a plant and haul them out to the job cheaper than it would have been to carry all of the component parts and the labor to these remote locations.

In the twentieth structure, the inherent

characteristics of prestressing itself made this method attractive. A bridge which was to be loaded with three feet of superimposed fill was bolstered up underneath with prestressed beams. The prestressing of the supplementary beams created a camber which brought them into firm contact with the original structure and enabled them to take their intended share of the increased load. This characteristic – that of being able to make the prestressed beam come up to and take its fair share of the load – is also being used in several other structures now being designed. This is far superior to wedging or dry-packing with grout.

As the use of prestressed concrete increased and was applied to a greater variety of bridge types, the question naturally arose as to how this new type of construction was comparing costwise with the conventional types we have used heretofore. To answer that question, this series of comparisons has been assembled. Comparisons are often misleading and great care must be taken lest false conclusions be drawn from the analysis of costs of structures in which conditions are not equal.

COST COMPARISONS WITH CONVENTIONAL DESIGNS

One of the difficulties in comparing prestressed concrete structures with conventional designs is that the prestressed type of design is often chosen to satisfy some particular condition for which there is no equivalent in conventional design. An example of this is a prestressed design chosen to save headroom in a separation structure. In the case of Hoppow Creek, for example (Plate 2), the use of prestressed design enabled the depth of girder to be reduced to 2'-2" for

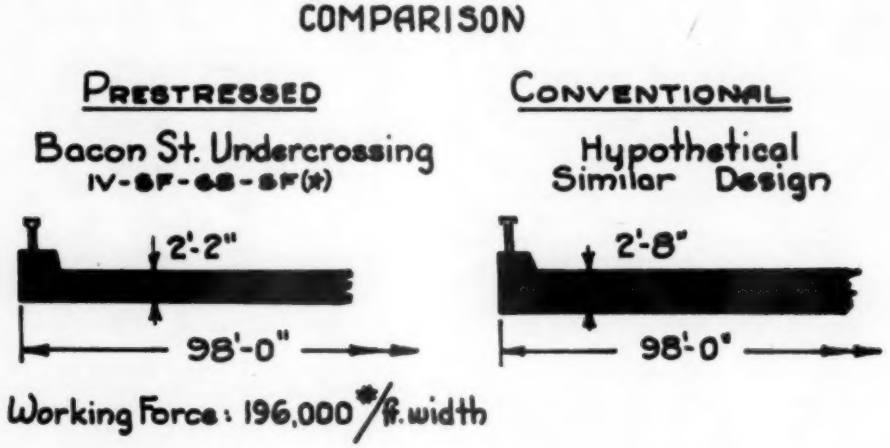


PLATE 1

the 50-foot spans. A conventionally designed T-beam for a 50-foot span would run 3'-6" to 4'-6" deep. Since the depth of girder is important at this location it becomes a question in making a fair comparison: If the prestressed girder could be made only 2'-2" deep should not the comparative cost of a conventional design also be figured upon a T-beam only 2'-2" deep? A comparison of this sort can not be fair. It does not present conventional concrete design in its most efficient form.

The comparisons made in this discussion are based upon the most economic use of the two relative designs. If the conventional design has a greater depth than the prestressed design, it is presumed that the approach grade line will have to be raised to compensate. Although it is realized that this raise in grade may introduce additional costs, chargeable to the conventional design, for purposes of comparison in this discussion only the actual cost of the two superstructures is considered.

COMPARISON

PRESTRESSED

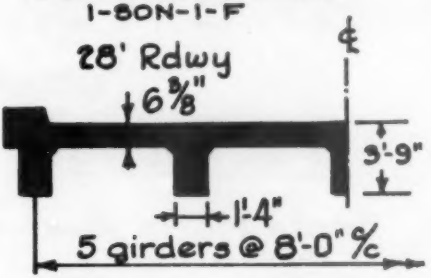
Hoppow Creek
I-DN-46-A Bids: 4-'55
w = 3'-7⅞" d = 2'-0"
s = 6" s_1 = 7" t = 5½"
Working Force = 360,000 #/gir.
Span = 50'
Area of deck = 1808 sq.ft.
Cost Superst = $14,950.
Cost per sq.ft. = $4.29 (adj)*

Route 20/1 Sep.
I-HUN-1 BIDS: 3-'55
w = 4'-8¾" d = 2'-6"
s = 7" s_1 = 8" t = 5¾"
Working force = 350,000 #/unit
Span = 46'
Area of deck = 1472 sq.ft.
Cost Superstr = $19,580.
Cost per Sq.ft. = $5.14 (adj)*

Dry Creek Canal
VI-FRE-125-FRE BIDS: 3-'55
w = 3'-9¼" d = 2'-0"
s = 7" s_1 = 9" t = 6"
Working Force = 374,000 #/gir.
Span = 44'
Area of deck = 3084 sq.ft.
Cost Superstr = $16,272.
Cost per Sq.ft. = $5.28

CONVENTIONAL

South Petaluma UC
I-SON-1-F



Twin bridges
Spans = 26', 52', 36'
Area of Deck = 4174 sq.ft.
Cost of Superstr. = $15,185.
Cost per Sq.Ft. = $3.63

Bids rec'd = 5-'54

* (adj) — Prices for structures in remote locations have been reduced to make them comparable to locations closer to labor and material sources.

PLATE 2

A number of the applications of prestressed principles in these first twenty structures built in California were of such a specialized nature that comparison to a conventional design is not possible. These have been eliminated from consideration. There are remaining twelve structures on which it is possible to make a cost comparison.

Nine of the structures are compared to conventional structures which are very closely parallel in span and site conditions. To make a comparison for the other three structures it is necessary to make a design for a hypothetical structure of the same span and width.

In making these comparisons, the construction cost for the superstructure only is used. Rail and curbing costs are not included. In order to obtain comparative cost per square foot the areas of the structure are taken to be the rail-to-rail width multiplied by the distance from paving-notch to paving-notch.

In the following comparisons a conventional design type is compared with one or more prestressed structures. The grouping is made so as to compare approximately equal span lengths. In order that only the relative initial structural costs may be considered, in the case where steel bridges are used as a comparison, the cleaning, painting and maintenance costs are not included. Where structures were built in remote locations, some adjustment was made in the cost in order to make them comparable to the cost of structures located closer to the sources of material and labor.

COST COMPARISONS

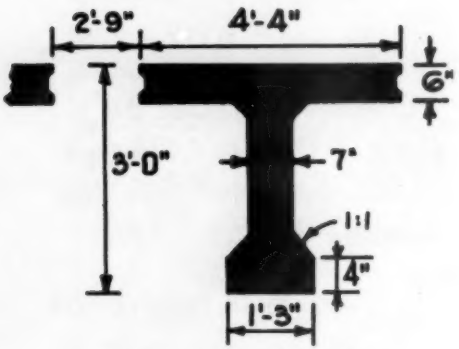
Slab Bridges

The Bacon Street Undercrossing (Plate 1) is a prestressed concrete slab built in

COMPARISON

PRESTRESSED

Santa Maria River
V-SLO-56-E

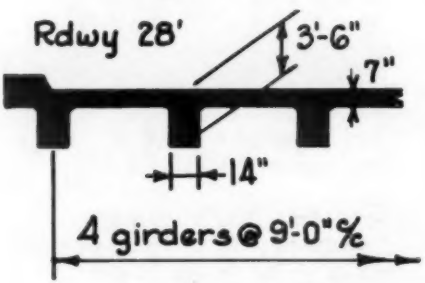


5 lines @ 7'-1" ℄

Working force: 353,000#/gir.

CONVENTIONAL

Santa Ana River
VII-ORA-175-B



Twin bridges

24 @ 50'-0"	SPANS	16 @ 50'-0"
38,196 sq.ft.	AREA OF DECK	55,384 sq.ft.
$134,960.	COST OF SUPERSTRUCTURE	$176,697
$3.53	COST PER SQ.FT.	$3.19

Bids rec'd - 7-'54

Bids rec'd - 7-'53

PLATE 3

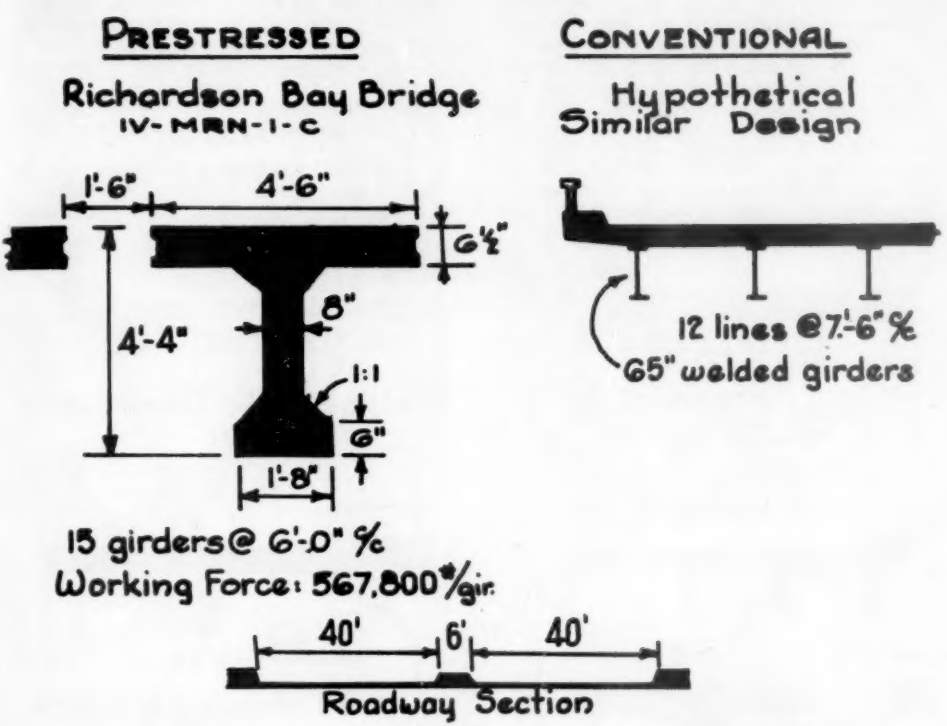


PLATE 4

San Francisco in 1953. The bridge is wider than it is long, having a 62-foot span with a width of 100 feet. The thickness of the slab is 2'-2". The slab was cast in place and was post-tensioned using wires grouped in sheaths. The slab was also prestressed laterally. The superstructure alone cost $40,580.00 for a deck area of 6,114 square feet. The unit cost of the structure was $6.65 per square foot.

Since no bridge of these dimensions of conventional concrete design was available, a hypothetical design was worked up for a 62-foot slab span reinforced in a conventional manner. The total cost of the superstructure was estimated at $40,250.00 for a similar deck area (6,114 square feet) making a total cost per square foot of $6.58.

The comparison shows the two types of designs to be almost identical in cost per square foot. However, it should be noted that the conventional design would have required 6 inches more depth of slab. This 6 inches extended to the approach fills would have materially increased the cost. Therefore, in this location the prestressed design was chosen in order to cut the required headroom to a minimum and keep the grades of the two roadways as close together as possible.

Channel Sections

In this comparison (Plate 2), three prestressed precast structures having a channel section are compared to a conventional design T-beam. Hoppow Creek was built along the sparsely populated Northern California

coast, not far from the ocean. The prestressed construction was chosen for three reasons. First, to get a minimum depth of superstructure so as to obtain as much waterway as possible without having to raise the roadway grade. Second, to obtain a structure which could be formed and poured where labor and materials were more plentiful and then hauled to the job. Third, to get a concrete structure which would require a minimum of maintenance in this damp location. Plate 2 shows the dimension of the section and the cost statistics of the structure. Because of the remote location of this job the prices were adjusted to make them comparable to locations closer to labor and material sources. The adjusted cost per square foot for this structure was $4.29.

The Route 20/1 Separation structure was also built on California's northern coast in 1955. The reasons for its choice were similar to those at Hoppow Creek. Plate 2 shows the details of the section and the adjusted price for this bridge was $5.14 per square foot.

The third structure in this group was the Dry Creek Canal Bridge near Fresno in California's warm Central Valley. Prestressed

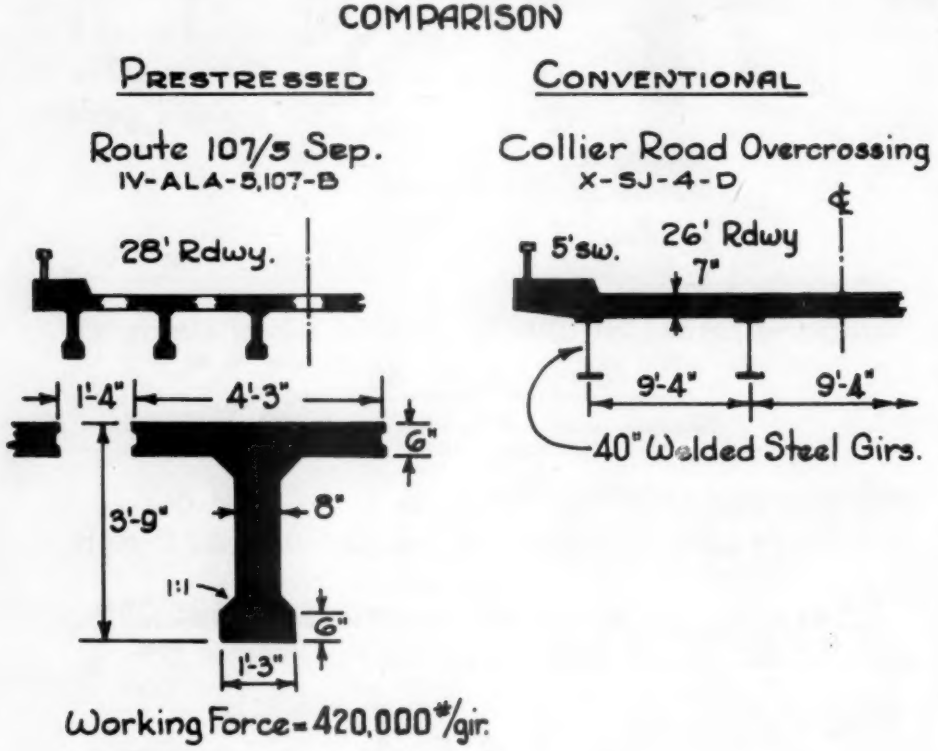


PLATE 5

COMPARISON

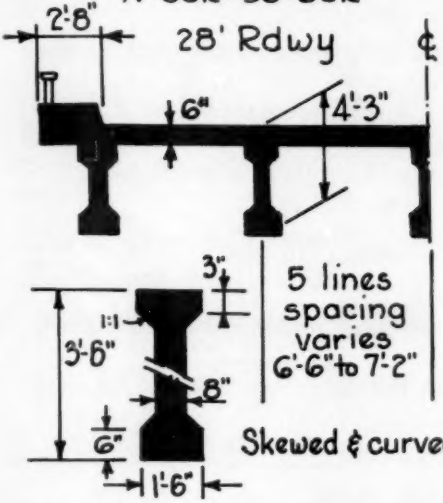
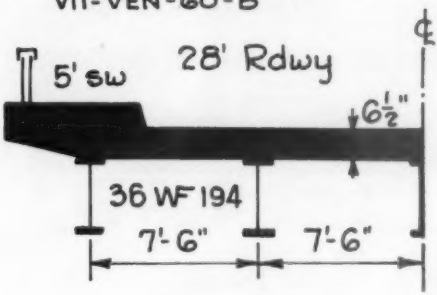
PRESTRESSED	CONVENTIONAL
Grant St. Under'xing IV-SCR-56-SCR	Frontage Rd. Over'xing VII-VEN-60-B
	
Working force - 348,000#/gir.	
53', 2@59', 33'	SPANS	2 @ 50'
5426 sq.ft.	AREA OF DECK	3718 sq.ft.
$27,977.	COST OF SUPERSTRUCTURE	$22,250.
$5.15	COST PER SQ. FT.	$5.98
Bids rec'd: 7-'55		Bids rec'd: 5-'54

PLATE 6

design was chosen to enable the units to be set in place over a full-flowing canal without the necessity of using falsework. The structure consisted of eighteen 44-foot precast channel sections bolted together and surfaced with an asphalt surfacing. The approximate weight of each unit was 16 tons. Plate 2 shows the data on the section and the cost per square foot was $5.28.

For comparison to the three precast and prestressed channel sections, a conventional T-beam design has been used. The South Petaluma Undercrossing, built in 1954, has five T-beams 3'-9" deep. The undercrossing consists of a pair of twin structures each of which is quite comparable to the three precast structures. The cost per square foot for this conventional T-beam worked out to be $3.63.

Although the unit cost of the T-beam is materially less than that of the precast and prestressed structures, it should be noted that there are other advantages which do not appear in the price, yet which are very beneficial to the construction. The cast-in-place T-beam required 1'-3" to 1'9" more depth of girder which decreased the headroom underneath the structure. In addition the falsework must be built for the cast-in-place structure, whereas the precast units may be cast in a yard and hauled to the site. Although the cost of the falsework would show up in the unit price, the advantages of construction without obstructing the chan-

nel or the roadway underneath are not always reflected in the unit prices.

PRECAST "T" SECTIONS

As shown on Plate 3, the Santa Maria River and the Santa Ana River bridges give a very good comparison of quite similar structures.

The Santa Maria River Bridge consisted of twenty-four 50-foot spans across a wide, flat gravel river bed. The T-section girders were cast and post-tensioned on the river bank, then hauled out and set into place. The saving in falsework was, to a degree, offset by the necessity of building a temporary casting yard on the river bank. One hundred twenty of the precast and post-tensioned girders were manufactured. The final unit cost for this superstructure was $3.53 per square foot.

The Santa Ana River consisted of a conventional T-beam design used for a pair of parallel bridges. The span lengths were the

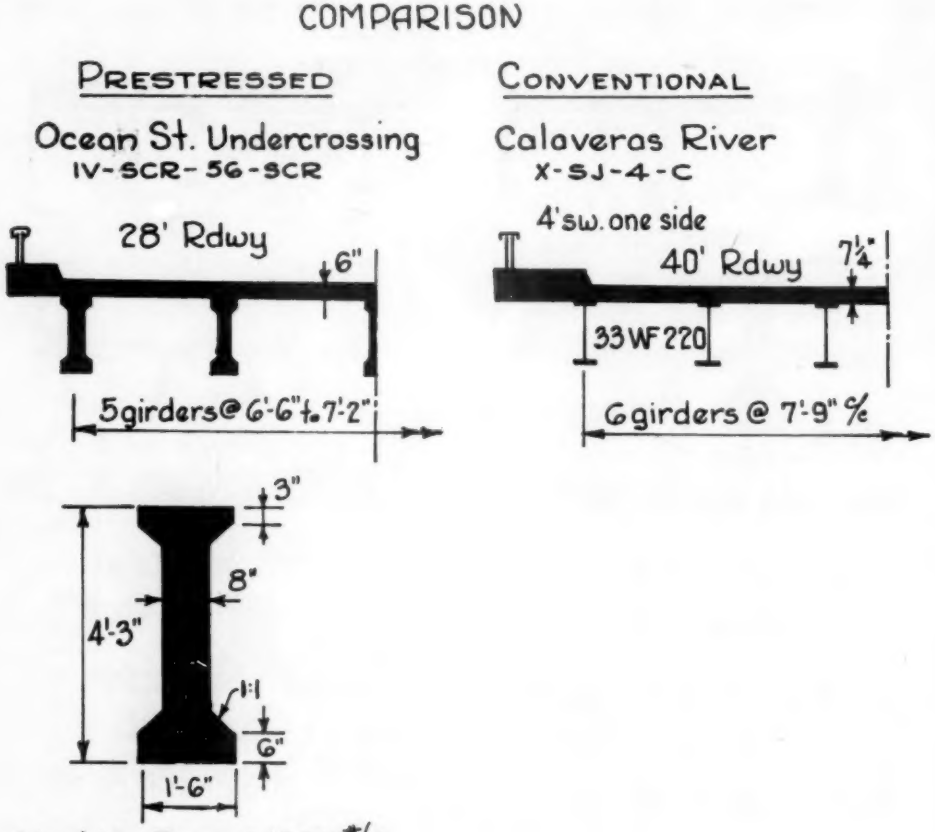


PLATE 7

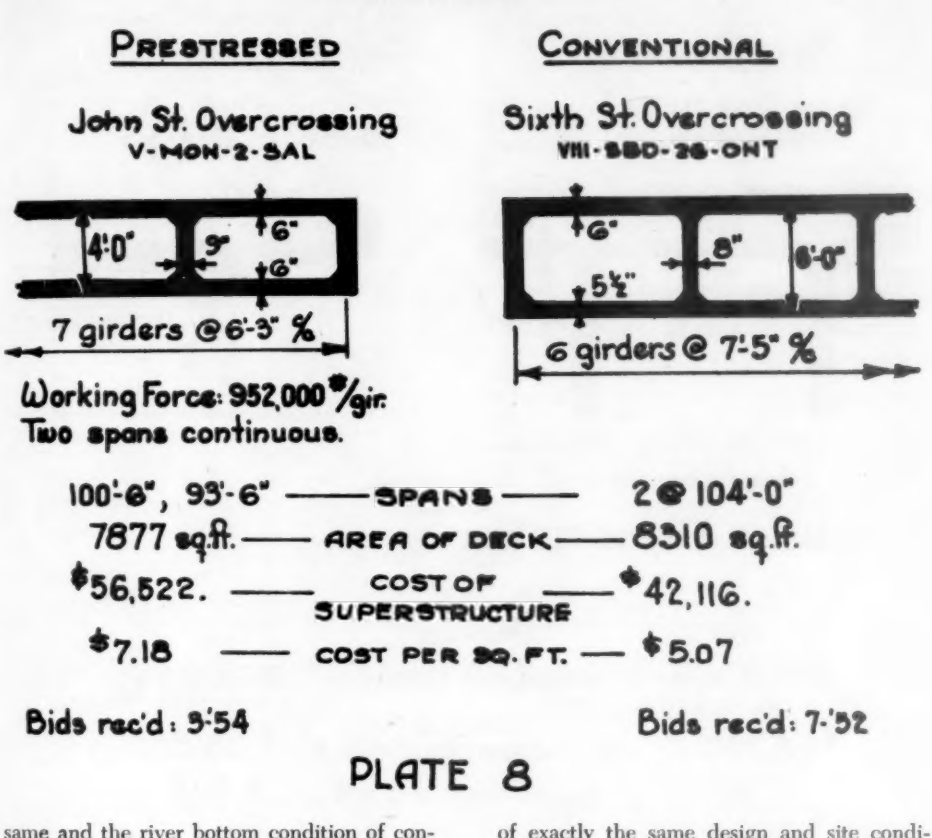


PLATE 8

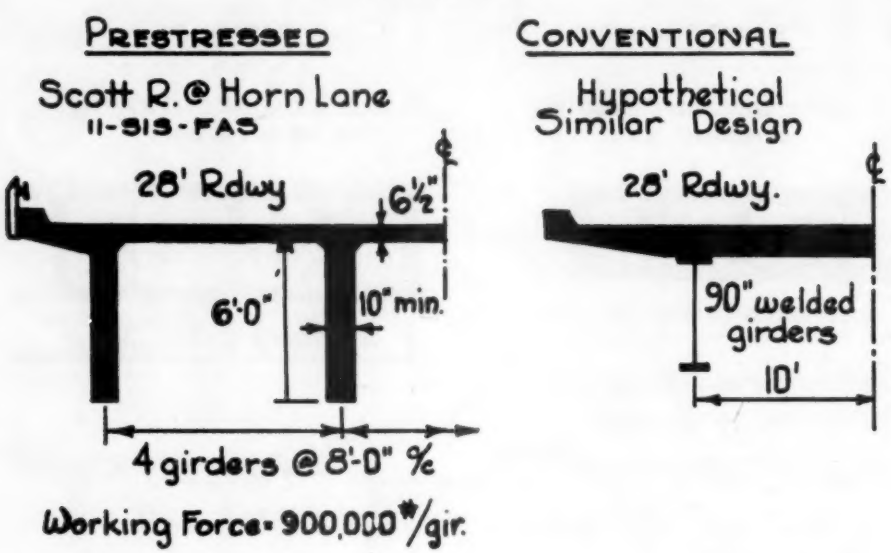
same and the river bottom condition of construction was very similar to that at Santa Maria River. The Santa Ana River possibly has a slight advantage in being somewhat closer to a large labor and material source. However, in this case the conventional T-beam design came out cheaper than the prestressed and precast design since the final figure for the Santa Ana River was $3.19 per square foot.

The Richardson Bay Bridge (Plate 4), afforded an opportunity to use a precast and prestressed design where the saving in falsework was very material. This bridge was built over an arm of San Francisco Bay in which the bottom was covered with a thick layer of thin mud. Falsework would have been prohibitively expensive. The portion of the structure over the water was designed for prestressed and precast beams which were lifted in place from a derrick barge. The beams were cast in a yard which had water access to San Francisco Bay and were hauled about 40 miles to the bridge site on barges. The operation worked very successfully and the unit cost for the precast portion of the bridge over the water was $3.94 per square foot.

Since there was no comparative structure of exactly the same design and site condition, a hypothetical design was made to compare to the Richardson Bay Bridge. It was presumed that because of the falsework difficulties it would be necessary to use steel welded girders even though in this salt water location that would increase the maintenance painting cost. Incidentally, those maintenance painting costs were not figured in the unit cost for this study, even though it is necessary that these costs be included in an overall economic study to select the type of structure to be used. The hypothetical design based upon prices for similar work now being realized, was $4.59 per square foot. This shows a material benefit in favor of the prestressed and precast construction.

Another example of a precast, prestressed T-section was the Route 107/5 Separation (Plate 5). In this case the prestressed section was used to obtain the minimum depth of girder and was precast to get the advantages of construction without falsework. The unit cost was $6.00 per square foot.

The Collier Road Overcrossing with welded steel girders and a concrete deck gave a comparison similar to the Route 107/5 Separation. The unit cost at $5.09 per square



foot, was considerably cheaper than the prestressed structure in this instance. The primary problem was that of erecting the structure over traffic and the T-beam design minimized the amount of concrete form work which had to be done over the traffic.

I-Section Precast Girders

Plates 6 and 7 show I-Section precast and prestressed girders compared with steel girders for comparable spans and conditions. In both of these cases the precast concrete beam is slightly deeper than its steel counterpart.

The Grant Street Undercrossing consisted of five girders 3'-6" deep with a maximum width of 1'-6". The beams were cast at the site and lifted into place. The unit cost was $5.15 per square foot for the superstructure.

The Frontage Road Overcrossing, using 36" wide flange steel beams, also had five girders and comparable spans. The cost of the superstructure was $5.98 per square foot.

The other precast prestressed I-section was the Ocean Street Undercrossing (Plate 7). Its cost of $4.60 per square foot was materially better than the cost of $5.00 per square foot which resulted on the Calaveras River Bridge. The Calaveras River Bridge was made up of six 33" wide flange steel beams and had a somewhat wider roadway. However, on a square foot basis the prestressed structure came out cheaper in both

of these cases. In neither instance was the depth of girder a critical factor.

As has been emphasized, these sections represent pioneering efforts on the part of both designers and construction forces. One evidence of the state of flux in this new field is the variation in dimensions of the "I," channel, and "T" sections. Obviously, such is not an economical practice. To encourage the semi-precision forms necessary in a concrete plant to produce these sections, it is essential that a standard section be adopted and used. After the period of experimenting with various sections as represented by the prestressed jobs discussed here, California is now standardizing these often-used sections.

BRIDGES PRESTRESSED IN PLACE

The comparison on Plate 8 shows a pair of concrete box girders, one of which was post-tensioned. The John Street Overcrossing was our first two span continuous prestressing job. The prestressing wires were run through the open cells. The depth of girder was of prime importance and the use of prestressed concrete enabled us to reduce the depth to only 4 feet. The job was built in 1954 and, being the first continuous structure of this sort in this area, it is felt that the price was raised somewhat as a result of uncertainty as to what the job would require. Unit cost was $7.18 per square foot.

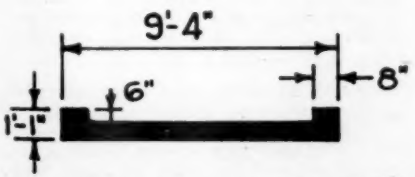
Compared to the prestressed box girder the Sixth Street Overcrossing built in 1952 gives a very good parallel comparison. Although the unit price at $5.07 is materially cheaper than the prestressed structure, it must be noted that the depth is also 2' greater. Also, because of the numbers of concrete box girders which had been built in the Los Angeles area, this type of construction was being done very economically. Though admittedly somewhat more expensive in this case, this comparison illustrates the fact that when a reduced headroom is essential, it is possible to obtain it at a price by the use of prestressed concrete. It is only fair to the prestressed girder to note that future designs of this type may well be considerably cheaper. The costs of the first bridge of a type are almost always somewhat high.

Incidentally, on the John Street structure, the difficulties during construction, which beset a contractor inexperienced in prestress-

COMPARISON

PRESTRESSED

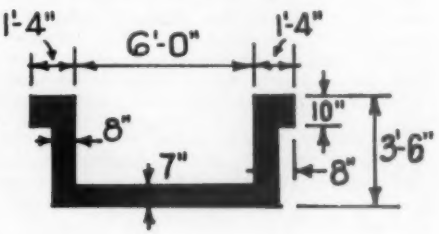
Harkness Ave. POC
IV-SF-68-SF



Working Force = 1,230,000#

CONVENTIONAL

Buhman Ave. POC
VII-LA-166-A



40'-58'-6"; 40' — SPANS — 2 @ 50'-8"

1779 sq.ft. — AREA OF DECK — 1656 sq.ft.

$5221.	COST OF SUPERSTRUCTURE	$5671.
$2.94	COST PER SQ. FT.	$3.42

Bids rec'd: 5-'54

Bids rec'd: 8-'52

PLATE 10

ing, bore out his apprehension. The fact that continuous spans are best stressed from both ends was not proved until a great deal of effort had been spent trying from one end. The grouting operation proved very troublesome and took weeks before it was successfully accomplished – largely because of lack of know-how. These experiences become known and other contractors become wary. Higher prices are the inevitable result.

Plate 9 shows a deep but thin section T-beam bridge which was poured in place and then post-tensioned. The six-foot-deep girders worked out very economically for this 120′ span. The unit cost was $7.32 per square foot. Because of the remote location of the structure it is estimated that this cost is about 7% higher than would be the cost of a structure closer to the labor and material markets.

No structure was available for comparison to the Scott River Bridge so a hypothetical similar design was worked up. Using two 90-inch welded girders with a similar span and width of roadway the unit cost was $9.12 per square foot, showing a material benefit on the side of the prestressed work.

Pedestrian Overcrossings

Plate 10 compares two pedestrian overhead structures. The rapid increase in the construction of freeways through heavily populated areas has greatly increased the number of structures which must be built for the passage of pedestrians. To be aesthetically attractive, a pedestrian structure must be light and airy in appearance. Prestressed concrete designs have gone a long way in improving the looks of pedestrian structures by making long thin members quite feasible. The Harkness Avenue Pedestrian Overcrossing is a simple slab span only 7″ thick with two shallow side beams only 13″ deep. The deck is surmounted by a 6-foot steel railing made up of parallel vertical bars. The supports are wall-type bents only 12″ thick. The whole effect of the structure is light and pleasing. The unit cost was $2.94 per square foot.

The conventional concrete design, the Buhman Avenue Pedestrian Overcrossing, also strove for a light thin appearance. However, because of the limitations of conventional design, it was necessary to make the edge beams 3′-6″ deep. The architectural treatment on the outside of the beam created a horizontal shadow which helped to break up the plain surface and a low railing similar to the Harkness Avenue structure contributed to the open feeling. However, either from the standpoint of appearance or from cost, at $3.42 per square foot, the conventional structure did not approach the clean open lines of the prestressed design.

SUMMARY

Comparisons of costs of materials and structures are at best difficult, confusing and contradictory. After years of experience with both concrete and steel structures, it is still difficult to get a true and fair comparison of these basic materials. With this in mind it is obvious that any comparison of prestressed concrete which has been in use a mere six years must obviously be found desirable in many ways. The facts presented in this paper bear this out. What conclusion may be then drawn?

CONCLUSION

In conclusion it is fitting to review certain facts connected with the development and use of prestressed concrete. Prestressed concrete is not a cure-all and is not applicable to all situations. It has certain advantages which, when utilized, may give benefits which will not be reflected in unit costs. It permits the construction of long beams which may be lifted into place without the necessity of falsework or interruption of traffic underneath a structure. It permits more slender and shallow members which allow greater headroom and decreases the required depth of a structure. It permits a member to be built in place and then post-tensioned to bring it up to take its full share of the load. When pre-tensioned, it permits members to be economically built in yards and hauled to the structure site.

There are other things which it will not do and places where it should not be used. It has little advantage on very short spans. It raises the difficulty of tying together precast concrete members. It calls for precision of workmanship beyond that usually found in concrete construction. It requires specialized knowledge and special techniques and equipment. It demands a certain amount of experience and know-how. This last reason has been outstanding in holding construction prices probably higher than warranted because of the timidity of contractors to undertake this new type of work.

Prestressed concrete is a new tool with definite and beneficial uses. To the engineer who considers it, cost alone is not the entire answer. It is true that as time goes on the cost of prestressed work will decrease. We, undoubtedly, will see a time when prestressed and precast beams can be bought on the market as a standard stock item. We shall probably see an amalgamation of the various prestressing systems to promote prestressed concrete as a whole and to subdue the competition which frequently acts to its detriment. All of these things will clarify the position of prestressed concrete in the construction field.

The time will soon come when the engineer who considers the use of prestressed concrete will find that for the benefits derived and for the money expended, prestressed concrete adds a new, economical and exciting tool to the many which the structural engineer may use.

The author is indebted to William Yusavage of the Department Pricing Section for compiling the comparative price figures.

What Feature* Is Common to Each of these Outstanding Concrete Structures?

PRESTRESSED CONCRETE PROJECTS:

1956 – **Lake Pontchartrain Bridge** – 24 miles of precast, prestressed deck spans – Palmer and Baker, Consulting Engineers

1956 – **Hangar at Hill Air Force Base** – 84 ton precast, prestressed girders spanning 130 feet – Roberts & Schaefer, Consulting Engineers

1956 – **Havana Sports Stadium** – 286′ thin shell dome with prestressed ring girder – Professor J. Vila and The Preload Company, Inc., Consulting Engineers

1955 – **Hampton Roads Project** – 2 miles of prestressed beam spans and prestressed piles – Parsons, Brinkerhoff, Hall & McDonald, Consulting Engineers

1954 – **Sunshine Skyway across Tampa Bay** – 3½ miles of prestressed beam spans – Parsons, Brinkerhoff, Hall & McDonald, Consulting Engineers

1953 – **Bridge at Heilbrun, Germany** – 353 foot span, world's longest prestressed span – Dr. Moersch, Consultant

1952 – **Office Building, Glenn L. Martin Co.** – 3 story precast, prestressed members bolted into place – Engineering Dept. of Glenn L. Martin Co., and The Preload Company, Inc.

1949 – **Walnut Lane Bridge, Philadelphia** – First large prestressed bridge in U.S. with a span of 155 feet – Engineering Dept. City of Philadelphia, The Preload Company, Inc. and Professor G. Magnel, Consultants

FLOATING CONCRETE STRUCTURES

1955 – **Tappan Zee Bridge, New York Thruway** – 6 buoyant boxes to support 80% of the weight of the main piers – Madigan-Hyland, Captain Emil H. Praeger, Consulting Engineers

1954 – **Kitimat Dock, British Columbia** – Cast on side and rotated 90° by shifting ballast – Frederick R. Harris, Consulting Engineers

1952 – **Pier 57, New York City** – 3 buoyant boxes with a total displacement of 73,000 tons floated 38 miles to permanent position at 15th Street, in New York City. Madigan-Hyland, Captain Emil H. Praeger, Consulting Engineers

1946 – **Floating Drydock, Uruguay** – Longest in the world

1943-44 – **Floating Drydock Program, U.S. Navy** – 13 large docks with a capacity of 2800 tons, and 2 smaller docks with a capacity of 400 tons

1942-45 – **Concrete Ship Program** – 125 ships and barges built for U.S. Maritime Commission during World War II

*The feature common to each of these projects is the use of Plastiment Concrete Densifier to obtain uniformity, uniform high strength and resistance to water and cracking.

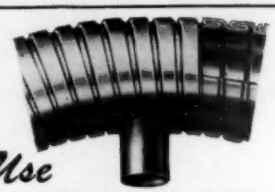
Of course, the great volume of Plastiment is consumed in every day concrete work such as conventional reinforced buildings, water reservoirs, and similar structures.

For a detailed explanation of Plastiment action in concrete, write today for the 8 page bulletin "Plastiment Concrete Densifier."

PLASTIMENT®
CONCRETE DENSIFIER

SIKA Chemical Corporation, Passaic, N. J.

DISTRICT OFFICES: Boston • Chicago • Dallas • Philadelphia Pittsburgh • Salt Lake City • Washington, D. C. • Montreal, Vancouver, Canada • Panama City, Panama • Dealers in Principal Cities



Use
STEELFLEX
ALL METAL *Flexible*
CABLE CASING
For Armoring Post Tension Reinforcing Assemblies

*Lengths to your specifications Flexible Tubing furnished with cement grouting inlets if specified.

Write for Bulletin U-100-BX-1

Quality ... ALL METAL FLEXIBLE HOSE PRODUCTS
UNIVERSAL METAL HOSE CO.
2187 S. KEDZIE AVE., CHICAGO, ILL.

P. C Items

Published monthly by
Prestressed Concrete Institute

This lively illustrated news bulletin presents news of Institute activities . . . views of members facilities . . . highlights outstanding prestressed concrete applications, and reports on latest developments in reserach, production of prestressed concrete.

Subscription $2.50 per year

To start your subscription, write to—

EDITOR
PCItems
3132 N.E. Ninth Street
Fort Lauderdale, Florida

advertising PROFESSIONAL service

LIBBY-BASKAM
CONSULTING ENGINEERS
Consulting Engineers
JAMES R. LIBBY ENIS Y. BASKAM
Prestressed Concrete Design • Plant Design
Bridges • Multi-Story Buildings • Stadiums
Industrial Buildings • Trestles • Reports
Consultation
151 Radcliff Dr., W. • E. Norwich, N. Y.

WAYLAN ENGINEERING CO.
Structural Consultants
A. M. OZELL WAYNE D. HEASLEY
PAUL M. ZIA, Consultant
Consultants in Prestressed Concrete
Reinforced Concrete • Steel Design
47 East Robinson Avenue
Orlando, Florida

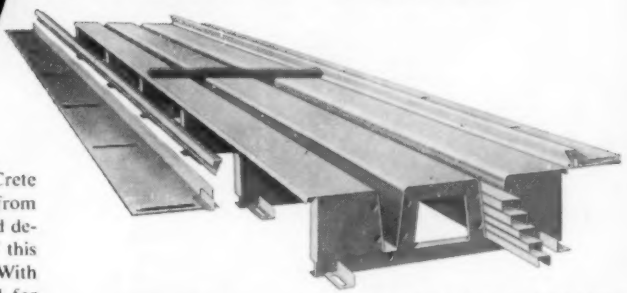
Lakeland Engineering
Associates, Inc.
Professional Engineers
P. O. Box 495 Phone Mutual 8-3971
Lakeland, Florida
Consultants in Prestressed Concrete Design

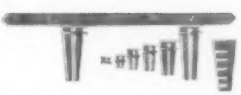
This PROFESSIONAL SERVICES *section has been established to enable engineers and architects to list the services they offer to the prestressing industry. Advertising rates are $10.00 per column inch. Copy should conform with ethical standards for advertising as established by the various professional associations which govern advertising of architects and engineers. Copy and inquiries should be directed to the* JOURNAL *Publication Office: 3132 N. E. 9th Street, Fort Lauderdale, Florida.*

MAKE PLANS NOW TO ATTEND THE WORLD CONFERENCE AND 3rd ANNUAL PCI MEETING, San Francisco, California, July 29 through August 2, 1957.

The choice of Prestressed Concrete Product Manufacturers

FORM-CRETE® STEEL FORMS





FORM-CRETE DOUBLE-T ALL-STEEL CASTING FORMS—*By utilizing a variety of gauge forms and stem heights an ingenious manufacturer can easily surpass the 37 individual prestressed concrete castings that we conservatively estimate may be produced from this one all-steel form assembly.*

The immediate acceptance and purchase of Form-Crete Steel Forms by prestressed concrete manufacturers from coast to coast provides ample assurance of the sound design, practical rugged construction and economy of this new innovation in the prestressed concrete field. With the addition of a line of I-beam forms engineered for casting all standard specified sizes of prestressed concrete I-beams we offer the most complete line of coordinated interchangeable forms and accessories obtainable anywhere. A new revised catalog is available soon—write for your copy today.

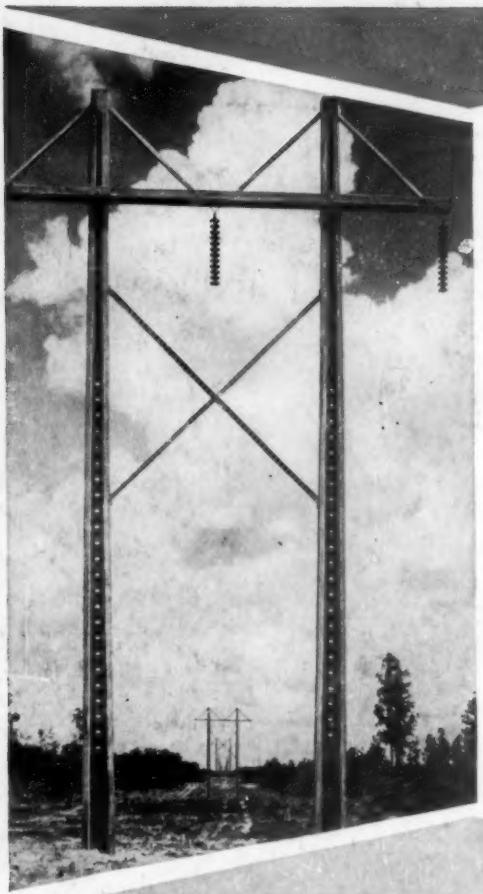
SEND FOR **FORM-CRETE BULLETIN 100-A**

PF-1

FOOD MACHINERY
AND CHEMICAL CORPORATION
FLORIDA DIVISION
LAKELAND, FLORIDA

FORM-CRETE STEEL FORMS FOR CASTING REINFORCED OR PRESTRESSED CONCRETE

DOUBLE "T" SLABS | SINGLE "T" JOISTS | HOLLOW AND SOLID LINTELS | SQUARE AND OCTAGONAL PILING | BRIDGE BEAMS | PAN TYPE BRIDGE DECKS



TODAY...

Roebling tensioning materials have created many markets for prestressed concrete . . . warehouses • garages • stadiums bridges • piers • hangars and other construction

TODAY...

Another new market for fabricators

Prestressed* Concrete Line Poles

Shown here is one of the latest in the constantly growing list of applications for prestressed concrete. These power line poles were designed and fabricated for the Florida Power Corporation by Finfrock Industries, Inc., Orlando, Florida.

Each pole is prestressed with $\frac{3}{8}''$ diameter Roebling uncoated 7-wire stress-relieved strands, pretensioned and bonded to the concrete.

Easily transported and installed, the 65 ft (above ground) poles have a life expectancy of 50 to 100 years. Once erected, they are maintenance-free. The high cost of replacement every few years is eliminated.

Why don't you investigate the possibilities of this new market to increase your profits? Roebling engineers, who pioneered in the development of prestressed concrete in America, will supply general information or help with specific problems. Write to Construction Materials Division, John A. Roebling's Sons Corporation, Trenton 2, N. J.

ROEBLING CF&I

A Subsidiary of The Colorado Fuel and Iron Corporation

*Not *to be confused with ordinary reinforced concrete*